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16. Abstract

Determining the relationships between pavement behavior and the design variables (pavement thickness, type of steel reinforcement and depth of steel reinforcement) was one main objective of a study Illinois started in 1960. The pavement behavior, expressed in terms of deflection, transverse cracking, distress, and riding quality was analyzed and correlated with the design variables. The data were collected from six experimental projects constructed throughout the State during 1963-66. The investigation also includes instruments and procedures used to expand the knowledge on stress levels and ranges in the steel and concrete of CRC pavement.

The findings indicate that the 7-in. and 8-in. (178-mm and 203-mm) CRC slab deflected greater than the edge of the standard 10-in. (254-mm) PCC pavement. The CRC slab thickness and the type of reinforcement have no effect on transverse cracking, but the depth of steel reinforcement has a major effect on transverse cracking. At the time of transverse crack development an abrupt change from compression to tension took place in the steel reinforcement.

In general, the performance of Illinois CRC pavements since opened to traffic (11 to 14 years of service) has been quite satisfactory. A few localized failures have developed and were found to be caused by construction deficiencies, poor subbase drainage, and a higher level of load applications.

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BEHAVIOR OF EXPERIMENTAL CRC PAVEMENTS IN ILLINOIS

By

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And

Richard K. Taylor

Interim Report
IHR-36
Continuously Reinforced Concrete Pavement

A Research Study Conducted by
Illinois Department of Transportation
Springfield, Illinois 62764
in cooperation with
U. S. Department of Transportation
Federal Highway Administration

March 1979

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

BEHAVIOR OF EXPERIMENTAL CRC PAVEMENTS IN ILLINOIS

INTRODUCTION

This report is a part of a continuing study conducted by the Illinois Division of Highways pertaining to the behavior and design study of continuously reinforced concrete (CRC) pavement (1, 2, 3, & 4). An adequate background for determining the most economical combination of pavement thickness, reinforcing steel type and amount and depth, correlated with traffic was not at hand. Therefore, in 1960 the Illinois Division of Highways began a comprehensive study of continuously reinforced concrete pavement.

During the development of the study a subcommittee of the Highway Research Board Committee on Rigid Pavements was actively engaged in correlating existing information on CRC pavements and in formulating recommendations on planning, design, and research for future experimental projects. With expectation of furnishing information not duplicated elsewhere, the work plan for the study was developed in accordance with the recommendations of the subcommittee.

The general objectives established for the study are, (1) to promote improved mechanized construction procedures for reducing the initial cost of CRC pavements, (2) to determine the significant relationships between pavement behavior and the design variables (pavement thickness, steel reinforcement type and depth, and subbase type and thickness), and (3) to evaluate various types of terminal joints or anchorages.

Provisions were made in the research work program to construct several experimental CRC pavements throughout the State. This provided an opportunity not only to obtain valuable constructive experience in several districts but also to evaluate certain design factors relative to pavement behavior under a variety of traffic and environmental conditions.

The design variables incorporated into the construction of the pavement within the scope of this report were the following:

(1) pavement thickness 7 in. (178 mm), and 8 in. (203 mm)

(2) depth of reinforcement 2 in. (51 mm), 3 in. (76 mm) and center of slab

(3) type of reinforcement; deformed bars and deformed fabric.

In 1963-66, an experimental project was constructed in each of six different highway districts. Except for one project that was carefully instrumented for intensive study, the remaining projects having one or two variables were constructed as observation pavements involving only routine observations, measurements, and tests. The observation pavements are 3.5 to 6.0 miles (5.6 to 9.7 km) long, and one or more of the design features (pavement thickness, type of reinforcement, or depth of reinforcement) is varied within each of the projects. Longitudinal reinforcement amounting to 0.6 percent of the cross-sectional area of the slab was used in all experimental sections. Each project also includes a section of conventional jointed pavement to serve as a control for comparing the performance of the pavements.

The instrumented pavement under intensive study, Project No. 4, is located near Springfield in District 6 and the project is identified as FA 196, Section 2-1, Sangamon County. This project contains most of the design parameters and limits these parameters to a common location which minimizes variations due to environment, traffic and construction practice. Instruments installed were strain gages, thermocouples and pavement-deflection measuring devices.

The design and construction details have been covered in an interim report (2).

The present report comprises an evaluation of deflection, transverse cracking and riding quality of 6 CRC and 5 PCC pavements constructed throughout the State during 1963-66. Also, this report describes the instruments and procedure used to expand the knowledge on stress levels and ranges in the steel and concrete of CRC pavement. The findings have indicated that the 7-in. (178-mm) and 8-in. (203-mm) CRC slab deflected greater than the edge of the standard 10-in. (254-mm) PCC pavement. The CRC slab thickness and the type of reinforcement have little, if any, effect on transverse cracking, but the depth of steel reinforcement has a major effect on transverse cracking. The depth of reinforcement not only affects the number of transverse cracks that developed in the pavement, but has a bearing on how tight the cracks are at the pavement surface and on the uniformity of the crack pattern.

At the time of transverse crack development in Project No. 4, an abrupt increase in tension took place in the steel reinforcement. Only a few strain gages continued to function effectively after the first year. Those few gages which continued to function for several years indicated a trend for the steel reinforcement to shift more toward compression with time. However, it is uncertain how much of the shift was due to drift of the instrumentation.

In general, the performance of Illinois CRC pavements since opened to traffic (11 to 14 years of service) has been quite satisfactory. A few localized failures have developed where full-depth patching was required. The failures were caused by construction deficiencies, poor subbase and subgrade drainage, and a higher level of load applications. The amount of patching appears to be related to the slab thickness, depth of reinforcement and accumulated loads. There is no trend, as far as riding quality is concerned, which can be associated with the type of pavement (CRC and PCC) and other incorporated variables.

LOCATION AND IDENTIFICATION OF EXPERIMENTAL PAVEMENTS

The research pavements were constructed in District 2, 4, 5, 6, 7 and 9. The geographical location of each test pavement is marked on the State map and reproduced as Figure 1. Projects 1, 2, 3, 4, 5 and 6 are located east of Silvis, east of Galesburg, west of Champaign, south of Springfield, east of Vandalia and east of Marion, respectively.

A listing of the test sites by location, with corresponding experimental features, is given in Table 1. All projects, with the exception of Project Number 4, have only one or two variables. Project Number 4 was carefully instrumented for intensive study and comparatively had more variables. A system for identifying each variable at each project was developed and is presented in the 7th column of Table 1. The first portion of this system identifies the Project Number and its subsection. The second portion identifies the type of pavement (CRC or PCC). The last portion of this system identifies the slab thickness (7 in. or 8 in.) (178 or 203 mm), type of reinforcement (fabric, bars) and depth of reinforcement (2 in.) (51 mm), (3 in.) (76 mm), and mid-depth. A control section of standard 10-in. PCC (marked as standard in the last portion of identification system) was constructed parallel or adjacent to the experimental pavement for behavior comparisons at all locations except for Project Number 4.

Project Number 4 was designed as a 2x2x2 factorial with two thicknesses of pavement - 7 and 8 in. (178 mm and 203 mm), two types of reinforcement - bars and deformed wire fabric, and two depths of steel placement - 2 in. (51 mm) below the surface and at mid-depth. Each main section is approximately 1200 ft (366 m) in length, and four of the eight test sections are replicated in shorter lengths. Experimental sections of this project are shown in Figure 2.

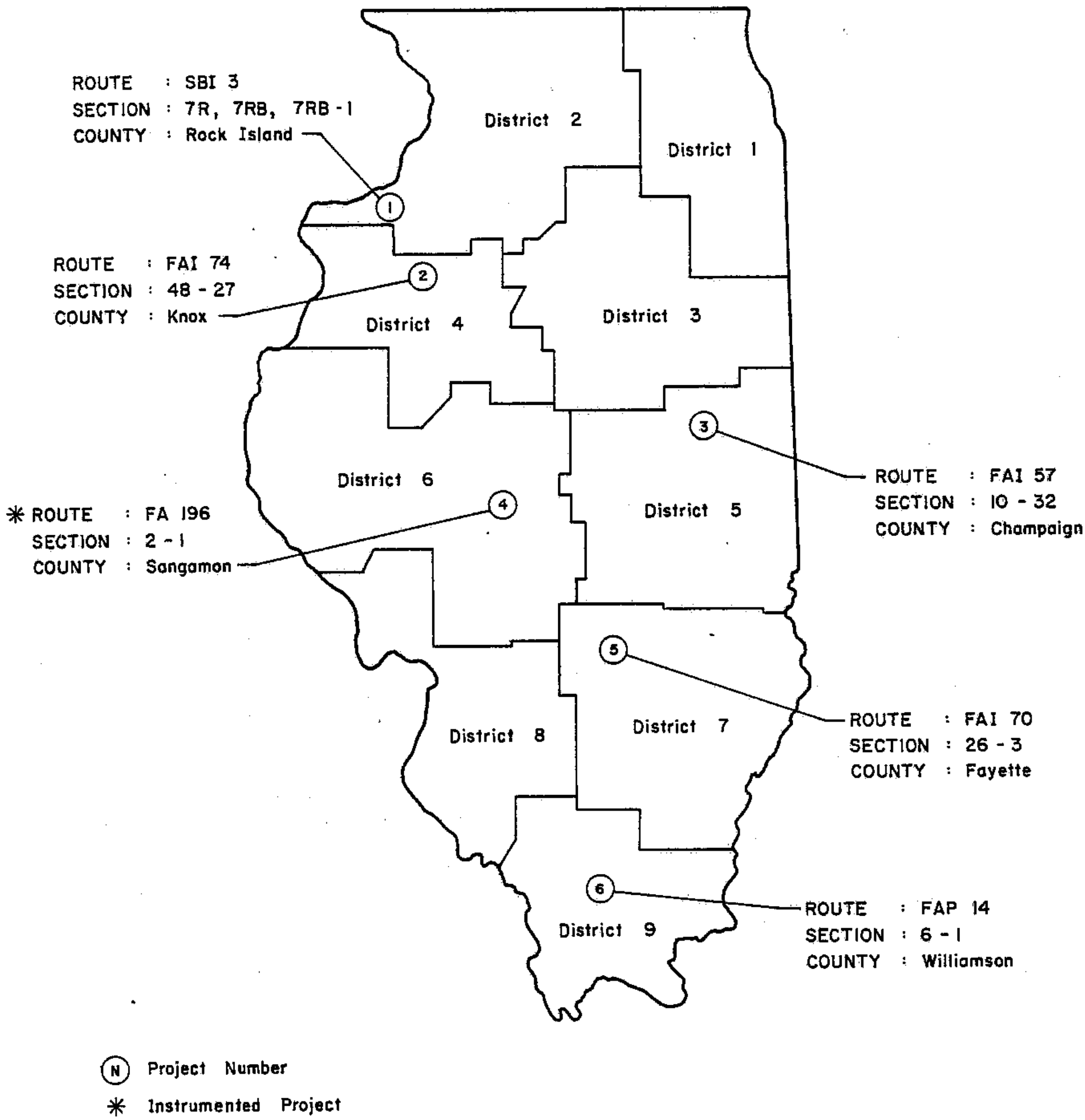


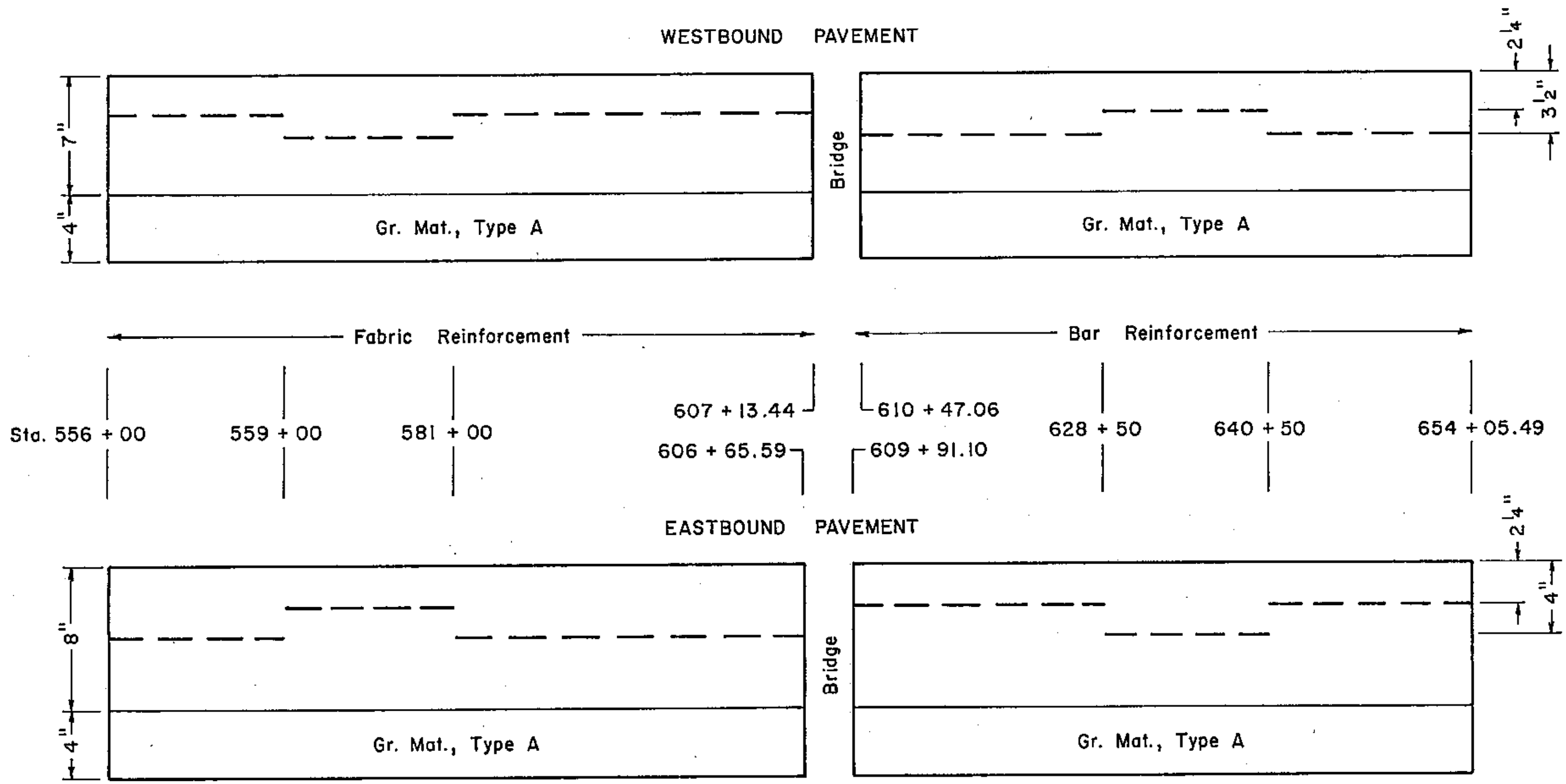
Figure 1. Location Of Experimental Projects.

TABLE 1

IDENTIFICATION OF EXPERIMENTAL PROJECTS AND DESIGN VARIABLES

						Experimental Feature			
						Test	Slab	Reinforcement	
Project Number	Route	Section Number	County	Year Built	Test Site Identification	Section mi.1/	Thickness in.2/	Type	Depth in.2/
1	SBI 3	7R-7RB	Rock Island	1964	1a-CRC-8f2	2.21	8	Fabric	2
				1964	1b-CRC-8f3	0.58	8	Fabric	3
				1964	1c-CRC-8fM	0.98	8	Fabric	4
				1964	1d-PCC-Std	3.77	10	-	-
2	FAI 74	48-27	Knox	1964	2a-CRC-7b2	2.04	7	Bars	2
				1963-64	2b-CRC-7bM	2.04	7	Bars	3.5
				1963	2c-CRC-7fM	1.45	7	Fabric	3.5
				1963-64	1d-PCC-Std	5.53	10	-	-
3	FAI 57	10-32	Champaign	1963	3a-CRC-7bM	2.01	7	Bars	3.5
				1963	3b-CRC-8bM	2.01	8	Bars	4
				1963	3c-PCC-Std	4.02	10	-	-
4	FA 196	2-1	Sangamon	1966	4b-CRC-7b2	0.23	7	Bars	2
				1966	4a-CRC-7bM	0.23	7	Bars	3.5
				1966	4d-CRC-7f2	0.23	7	Fabric	2
				1966	4c-CRC-7fM	0.23	7	Fabric	3.5
				1966	4e-CRC-8b2	0.23	8	Bars	2
				1966	4f-CRC-8bM	0.23	8	Bars	4
				1966	4g-CRC-8f2	0.23	8	Fabric	2
				1966	4h-CRC-8fM	0.23	8	Fabric	4
5	FAI 70	26-3	Fayette	1963	5a-CRC-8b2	2.00	8	Bars	2
		26-4		1963	5b-CRC-8b3	2.00	8	Bars	3
				1963	5c-CRC-8b3	2.00	8	Bars	4
				1963	5d-CRC-Std	6.00	10	-	-
6	FA 14	6-1	Williamson	1964	6a-CRC-7f2	1.90	7	Fabric	2
				1965	6b-CRC-7fM	2.09	7	Fabric	3.5
				1965	6c-PCC-Std	2.09	10	-	-

Note: 1/ 1 mile = 1.61 km
2/ 1 inch = 25.4 mm



Note : ——— Depth of reinforcement from top of slab to center of longitudinal steel.
1 inch = 25.4 mm

Figure 2. Experimental Sections Of CRC Pavement In District 6.

DEFLECTION

The structural response of CRC and PCC pavements to load as measured by deflection has been investigated in Illinois. It is generally believed that the deflection of a pavement is influenced by load, subgrade support, total thickness of pavement structure, intrinsic properties of pavement structure, crack spacing, crack width, percentage of longitudinal reinforcement and environment. The influence of slab thickness, type of pavement (CRC and PCC), depth of reinforcement, type of reinforcement, effective crack spacing, pavement age and environment on the pavement's structural response, as measured by static deflection, was investigated and will be discussed in this section.

The static deflection measurements were obtained during early spring of 1970 from 6 experimental pavements constructed throughout the State during 1963-66. In addition, spring and fall measurements - from April 1970 to April 1974 - were obtained from the instrumented pavement (Project No. 4). The deflections were taken by positioning a Benkelman Beam on the shoulder of the roadway at an angle of 30 degrees to the longitudinal edge of the pavement slab with the probe stationed on the pavement about one inch from the pavement edge pointing toward the truck. Before taking deflection readings, the pavement at each test location was "ironed out" by making four passes with the test truck. The truck was a single-axle type and was loaded with rear-axle loading of 18,000 lbs. The tire inflation pressure was 75 lbs per square inch. The center of the dual tires on the right side of the truck was kept 20 in. (508 mm) from the edge of the pavement. About seven readings per two-lane mile were taken, with all readings for CRC pavements taken at transverse crack positions while corner and edge positions were measured on the PCC pavements.

Slab Thickness and Type of Pavement

The average deflection for each CRC and PCC pavement is shown in Tables 2 and 3, respectively.

To evaluate the influence of CRC slab thickness on static deflection an analysis was made by comparing deflection measurements from sections (Project No. 3 & 4) that have identical subbases, subgrade soil and environmental conditions. The result of this analysis is shown in Figure 3. As expected, on the whole, 7-in. (178-mm) CRC slab deflected more than 8-in. (203-mm) CRC slab. The numerical difference in the amount of deflection varies from 0.013 in. (0.33 mm) to 0.024 in. (0.60 mm).

A control section of standard 10-in. (254-mm) PCC pavement was constructed parallel or adjacent to the experimental pavement for behavior comparisons of all locations except for Project No. 4. To evaluate the influence of pavement type on the static deflection, an analysis was made by comparing deflection measurements of CRC and PCC pavement for each project. As stated previously, deflection measurements for CRC pavements were taken at transverse cracks. Even though the boundary conditions for load distribution for CRC at transverse cracks are not precisely the same as for PCC at the edge or corner, a comparison of deflection measurements still helps to evaluate the influence of pavement type. The results are presented in Table 4. As can be seen, CRC pavement deflected greater than the edge of standard PCC pavement. Seven- and eight-inch (178-mm and 203-mm) CRC pavement deflected 3 to 6 and 1 to 4 thousandths of an inch, (0.08 to 0.15 and 0.03 to 0.10 mm), respectively, greater than the edge of standard PCC pavements. The difference is relatively greater for 7-in. (178-mm) pavement than for 8-in. (203-mm) pavement. From this, the effect of slab thickness on the edge deflection is apparent.

TABLE 2

SUMMARY OF DEFLECTIONS
FOR
EXPERIMENTAL CONTINUOUSLY REINFORCED PAVEMENTS-
APRIL 1971

Region or District	Test Site Identification	Slab Thickness (in.)*	Reinforcement		No. of Tests	Deflection	
			Type	Depth (in.)*		Average $x10^{-3}$ (in.)*	Standard Deviation
2	1a-CRC-8f2	8	Fabric	2	16	9	3.2
	1b-CRC-8f3	8	Fabric	3	6	12	1.4
	1c-CRC-8f4	8	Fabric	4	7	13	2.3
4	2a-CRC-7b2	7	Bars	2	16	14	3.7
	2b-CRC-7bm	7	Bars	3.5	7	15	2.8
	2b-CRC-7bm	7	Bars	3.5	8	15	2.0
	2c-CRC-7fm	7	Fabric	3.5	11	14	3.0
5	3a-CRC-7bm	7	Bars	3.5	16	17	4.1
	3b-CRC-8bm	8	Bars	4	15	13	3.7
6	4a-CRC-7b2	7	Bars	2	4	20	4.0
	4b-CRC-7bm	7	Bars	3.5	6	24	3.8
	4c-CRC-7f2	7	Fabric	2	-	-	
	4d-CRC-7fm	7	Fabric	3.5	-	-	
	4e-CRC-8b2	8	Bars	2	6	16	3.6
	4f-CRC-8bm	8	Bars	4	5	13	2.5
	4g-CRC-8f2	8	Fabric	2	5	15	1.0
	4h-CRC-8fm	8	Fabric	4	5	14	0.7
7	5a-CRC-8b2	8	Bars	2	16	15	2.4
	5b-CRC-8b3	8	Bars	3	12	16	2.8
	5c-CRC-8bm	8	Bars	4	14	12	1.6
9	6a-CRC-7f2	7	Fabric	2	10	11	4.3
	6b-CRC-7fm	7	Fabric	3.5	15	12	2.5

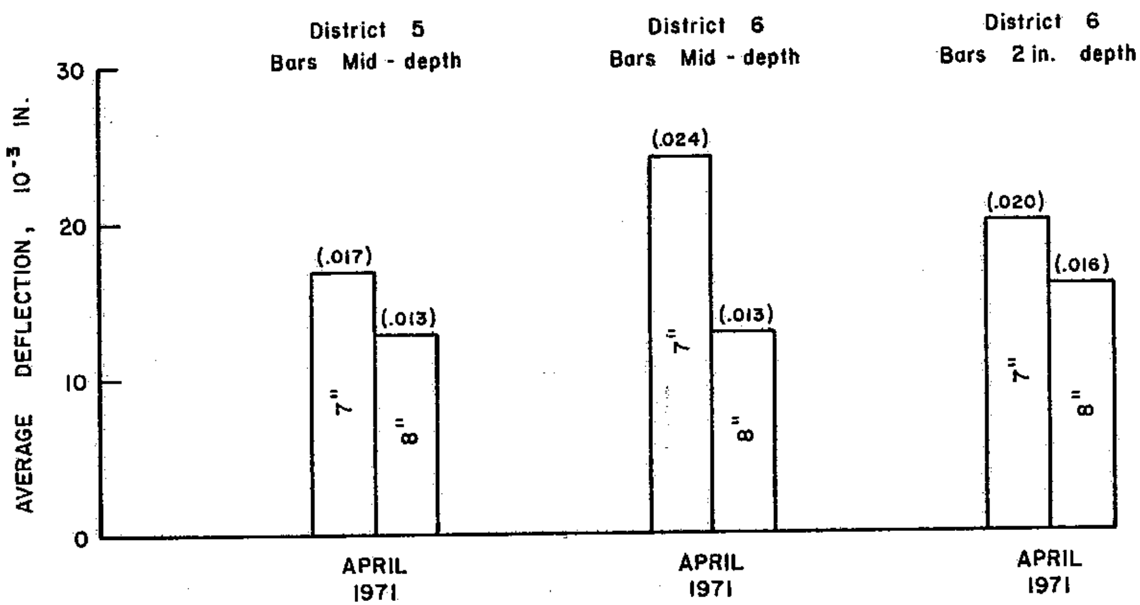
* 1 inch = 25.4 mm

TABLE 3

SUMMARY OF CORNER AND EDGE DEFLECTIONS
FOR
STANDARD PCC PAVEMENTS -
APRIL 1971

Region or District	Test Site Identification	Corner Deflection Average $\times 10^{-3}$ (in.)*	Corner Deflection Standard Deviation	Edge Deflection Average $\times 10^{-3}$ (in.)*	Edge Deflection Standard Deviation
2	1d-PCC-Std	10.6	2.5	7.0	1.7
4	2d-PCC-Std	16.3	3.7	11.1	2.1
5	3c-PCC-Std	11.5	2.7	11.7	2.9
7	5d-PCC-Std	16.5	7.0	10.7	3.1
9	6c-PCC-Std	10.0	3.5	8.3	1.6

* 1 inch = 25.4 mm



Note : 1 inch = 25.4mm

Figure 3. Relationship Between CRC Slab Deflection And Slab Thickness.

TABLE 4

INFLUENCE OF PAVEMENT TYPE
ON DEFLECTION

District	Slab Thickness in.* CRC	Slab Thickness in.* PCC	Average Deflection in.* CRC	Average Deflection in.* PCC Edge	Average Deflection in.* PCC Corner	Difference in Average Deflection in.* CRC Minus PCC Edge	Difference in Average Deflection in.* PCC Corner Minus CRC
4	7	10	.014	.011	.016	.003	.002
5	7	10	.017	.011	.012	.006	-.005
9	7	10	.012	.008	.010	.004	-.002
2	8	10	.011	.007	.011	.004	.000
5	8	10	.013	.012	.011	.001	-.002
7	8	10	.014	.011	.017	.003	.003

Note: * 1 in. = 25.4 mm

There is no apparent trend between the CRC pavement deflection and the PCC deflection. Corner deflection of the PCC pavement was greater than the CRC pavement for 3 projects and smaller for 3 projects.

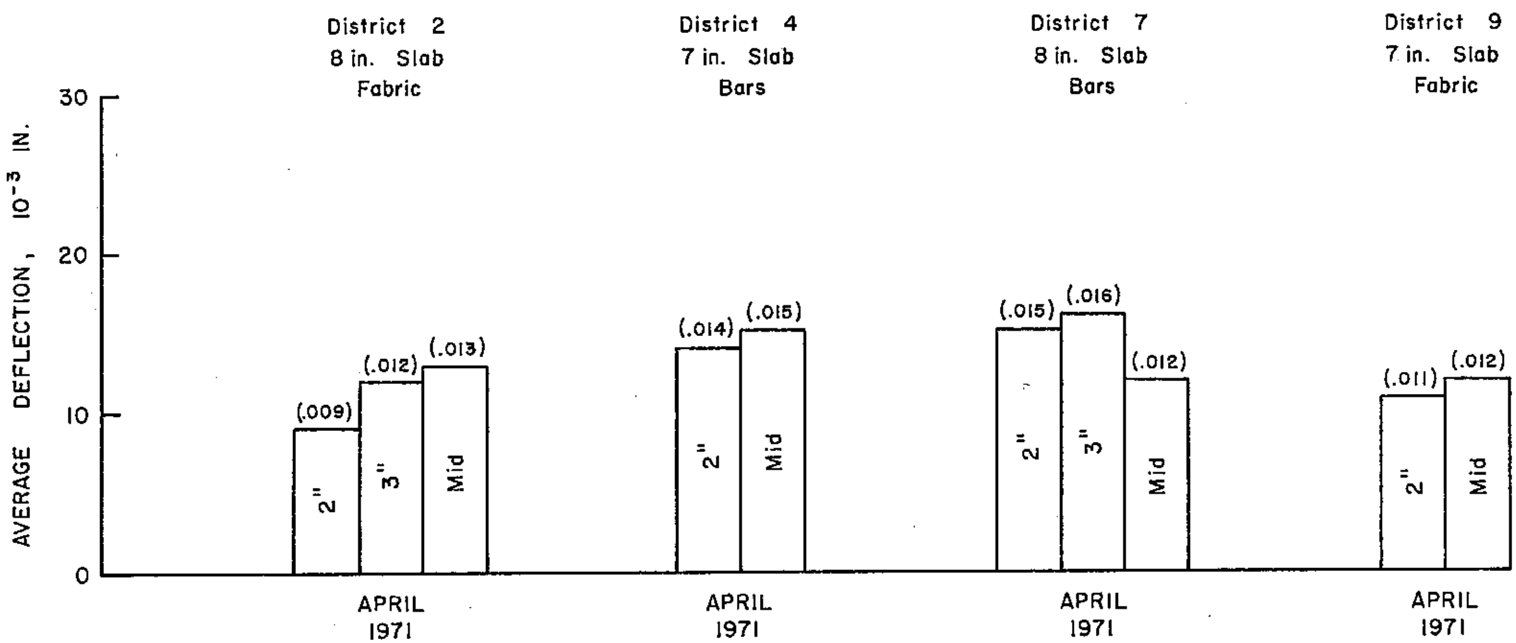
Depth of Reinforcement

The data were processed to find if there exists a relationship between the depth of reinforcing steel and the average CRC slab deflection. Findings are shown in Figure 4 and 5. The reinforcement depth is measured from the pavement surface to the upper surface of reinforcement for the 2- and 3-in. (51- and 76-mm) depth, and from the pavement surface to the center of reinforcement for the steel placed at mid-depth. Two variable depths, mainly 2-in. (51-mm) and mid-depth, were incorporated during the construction of 5 experimental pavements. On the remaining 2 experimental pavements, which are 8 in. (203 mm) thick, the steel was placed at 2 in. (51 mm), 3 in. (76 mm) and at mid-depth. The governing specifications permitted a construction tolerance of plus or minus 1/2 in. (4 mm) vertically from the design locations.

The findings from 4 out of 7 experimental pavements indicate that the average deflection increased as the depth of reinforcement increased from 2 in. (51 mm) to mid-depth. This increase is from 1 to 4 thousandths of an inch (0.03 to 0.10 mm). For the remaining 3 experimental pavements the average deflection decreased as the reinforcement depth increased from 2 in. (51 mm) to mid-depth. This decrease is from 1 to 3 thousandths of an inch (0.03 to 0.08 mm). These findings suggest that there is no general trend between CRC slab deflection and the depth of reinforcement.

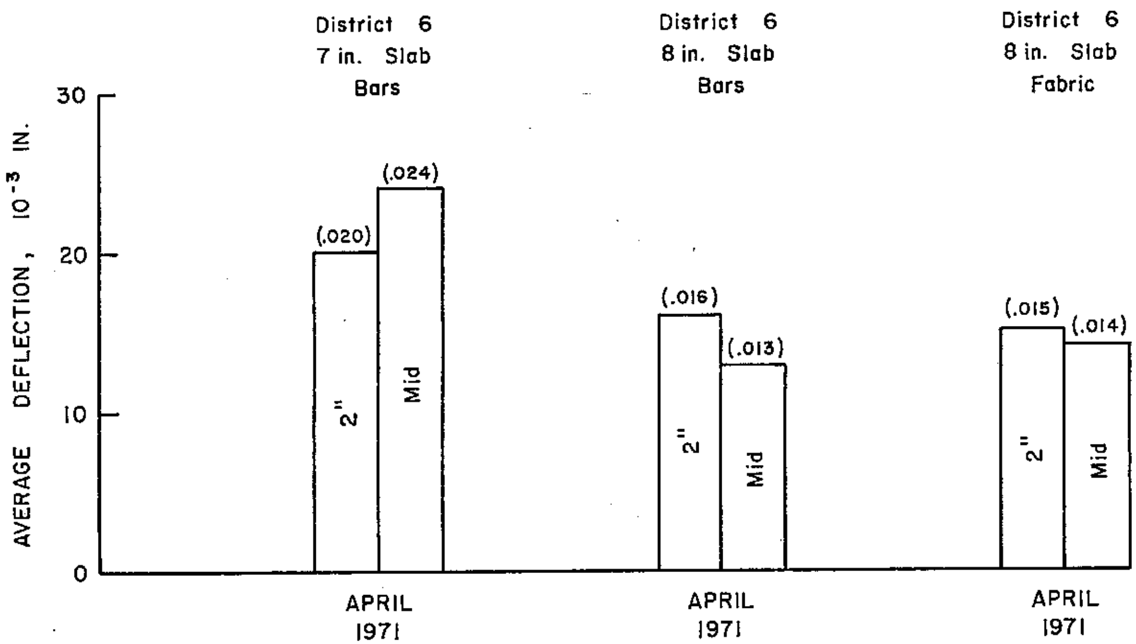
Type of Reinforcement

Figure 6 is a graph showing the average deflections relative to the type of reinforcement used in the construction of the CRC pavements. The projects for which the deflections are reported were selected so that, as far as possible, only one variable was present and evaluated. As can be seen from this figure,



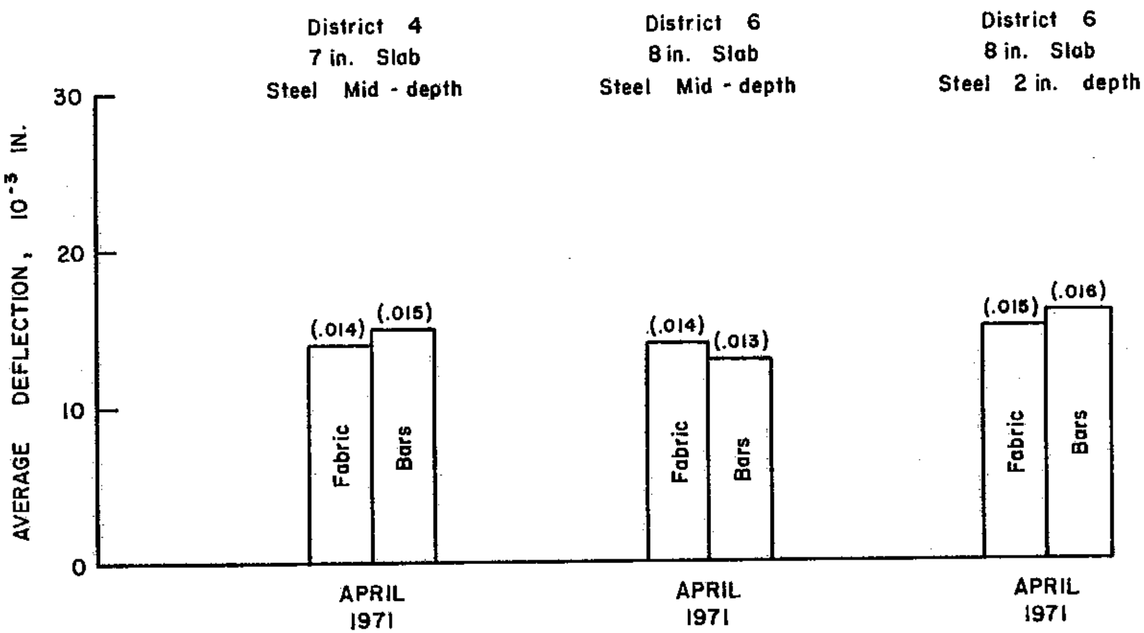
Note : 1 inch = 25.4 mm

Figure 4. Relationship Between CRC Slab Deflection And Depth Of Reinforcement.



Note : 1 inch = 25.4mm

Figure 5. Relationship Between CRC Slab Deflection And Depth Of Reinforcement For District 6.



Note : 1 inch = 25.4mm

Figure 6. Relationship Between CRC Slab Deflection And Type Of Reinforcement.

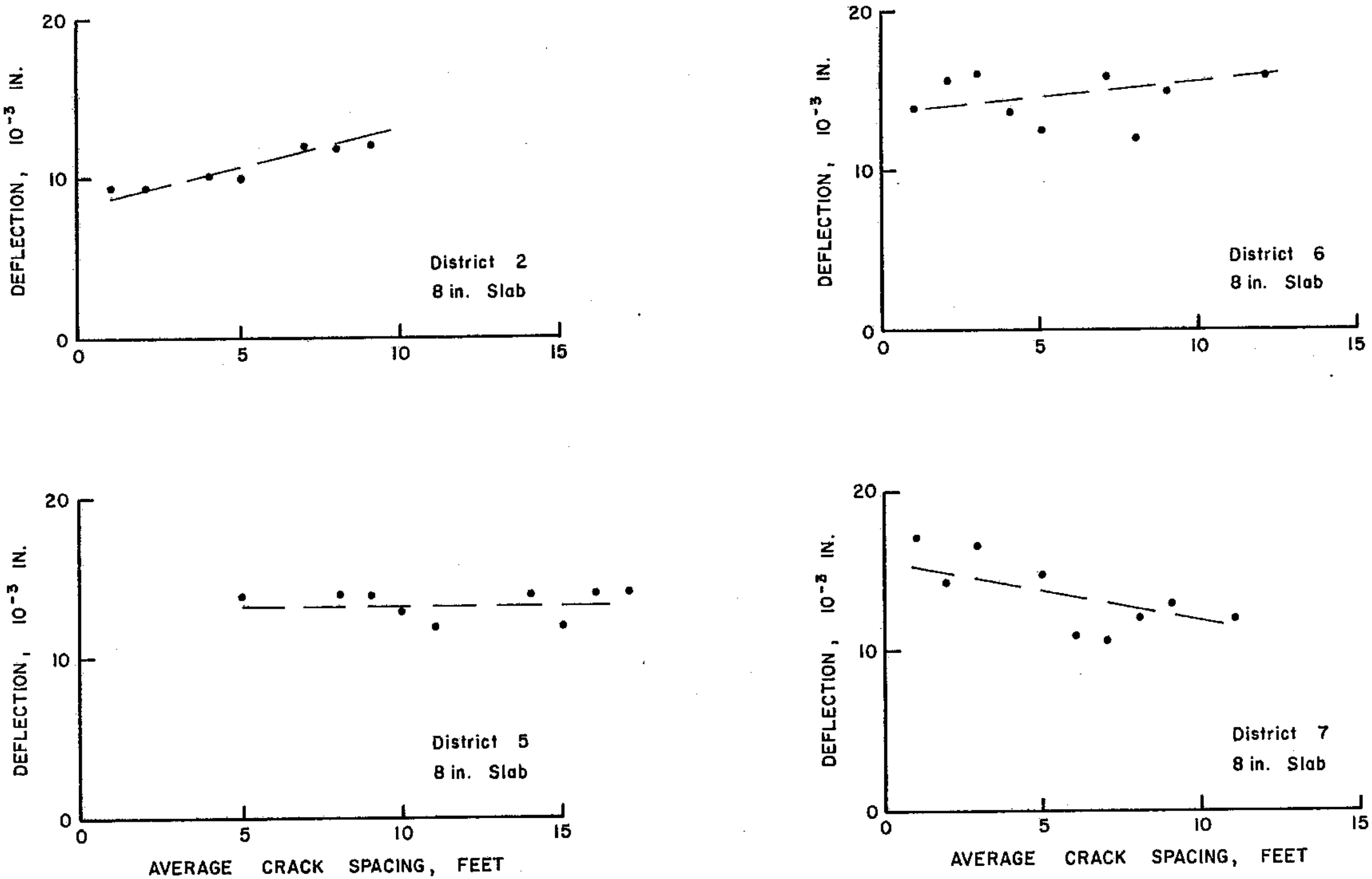
there is no pronounced trend or difference in the average deflections due to the type of reinforcement.

Effective Crack Spacing

Figure 7 is an example of a relationship between deflection and effective crack spacing for an 8-in. (203-mm) CRC pavement. The effective crack spacing is an indication of the length of concrete which is affecting the movement at a crack. The effective crack spacing is one-half the distance between the crack where the deflection is taken and adjacent cracks on both sides. On this graph a point represents the average value for a given effective crack spacing. As can be seen from this figure, there is no trend between the deflection and the effective crack spacing for 8-in. (203-mm) CRC pavement.

Pavement Age

The instrumented pavement (Project No. 4) is the only pavement in this study for which long-term deflection data have been collected. This project contained most of the design parameters, and limits these parameters to a common location which minimizes variation due to environments, traffic and construction practice. The pavement of this project was placed during the fall of 1966. Very limited deflection data were collected prior to April 1970, but considerable deflection data were collected from April 1970 to April 1974. At a given test location, the deflection was measured every time at approximately the same time of the day. A comparison of the change in deflection level with time or pavement age for the instrumented project is given in Table 5. The average deflection values from these pavements are somewhat greater than the identical design pavements constructed in other parts of Illinois. Some light pumping and distress were observed within two years after construction. This condition was more pronounced for 7-in. (178-mm) CRC pavement than for 8-in. (203-mm) CRC pavement.



Note : 1 inch = 25.4 mm
1 foot = 0.305 m

Figure 7. Relationship Between CRC Slab Deflection And Crack Spacing.

TABLE 5

SUMMARY OF DEFLECTIONS RELATIVE TO AGE FOR INSTRUMENTED CRC PAVEMENTS

Year of Observation	Precipitation & Departure from Normal 1/ (in.)*		Average Static Rebound Deflection (in.)*					
			7-inch Slab			8-inch Slab		
	Precip.	Departure	April	Oct.	Oct. Def. - April Def.	April	Oct.	Oct. Def. - April Def.
1970	1.99	-.89	.021			.014		
	7.73	+4.80		.026	+.005		.021	+.007
1971	1.18	-1.70	.022			.014		
	3.76	+ .83		.020	-.002		.013	-.001
1972	4.03	+1.15	.019			.015		
	3.95	+1.02		.029	+.010		.016	+.001
1973	7.89	+5.01	.032			.022		
	3.28	+ .35		.022	-.010		.014	-.007
1974	3.39	+ .69	.020			.015		

1/ 30 days prior to deflection measurements

*Note: 1 inch = 25.4 mm

The problem appears to be primarily associated with a condition of unusually poor subgrade drainage. To prevent severe pavement distress, which was imminent, from developing fairly early in the service life of the pavement, a positive underdrain system along the outside shoulders immediately adjacent to the pavement edge was installed during the fall of 1972.

Also, in this table an apparent increase in the average deflections can be noticed for October 1970, October 1972 and April 1973. Rainfall recorded at Springfield Airport 30 days prior to deflection measurements is given in the 2nd and 3rd columns of this table. Precipitation for September 1970 was 7.73 in. (196 mm), which is 4.80 in. (122 mm) above normal. Precipitation for September 1972 was 3.15 in. (80 mm) which is 1.02 in. (26 mm) above normal. The rainfall for March 1973 was 7.89 in. (200 mm) which is 5.01 in. (127 mm) above normal. Two weeks prior to each deflection reading, rainfall was above normal and this, in addition to poor subgrade drainage, may have contributed to high deflections. These higher deflection measurements are very critical from a stress point of view, but are special cases and are not representative of normal conditions; therefore, readings from these three periods will not be included in the evaluation relative to age.

The general level of deflection measured during April or October over a period of several years did not change appreciably. For the 7-in. (178-mm) slab, the average deflection measurements taken during April of each year fluctuate between 0.019 in. and 0.022 in. (0.48 mm and 0.56 mm). For October, their fluctuation is between 0.020 in. and 0.022 in. (0.51 mm and 0.56 mm). The average deflection of the 8-in. (203-mm) slab taken during April was between 0.014 in. and 0.15 in. (0.36 mm and 0.38 mm). The values for October are very similar.

Another relationship, namely, the influence of CRC slab thickness on static deflection, is apparent from this table. As expected, the 7-in. (178-mm) CRC slab deflected more than the 8-in. (203-mm) CRC slab. The seven-in. (178-mm) slab deflected approximately 0.006 in. (0.15 mm) more than the 8-in. slab. This difference in the average deflections did not change for April or October measurements.

Deflection measurements obtained during April are different than those obtained during October, but there is no trend which suggests that April or October deflections are always higher. April deflections are sometimes higher and sometimes lower.

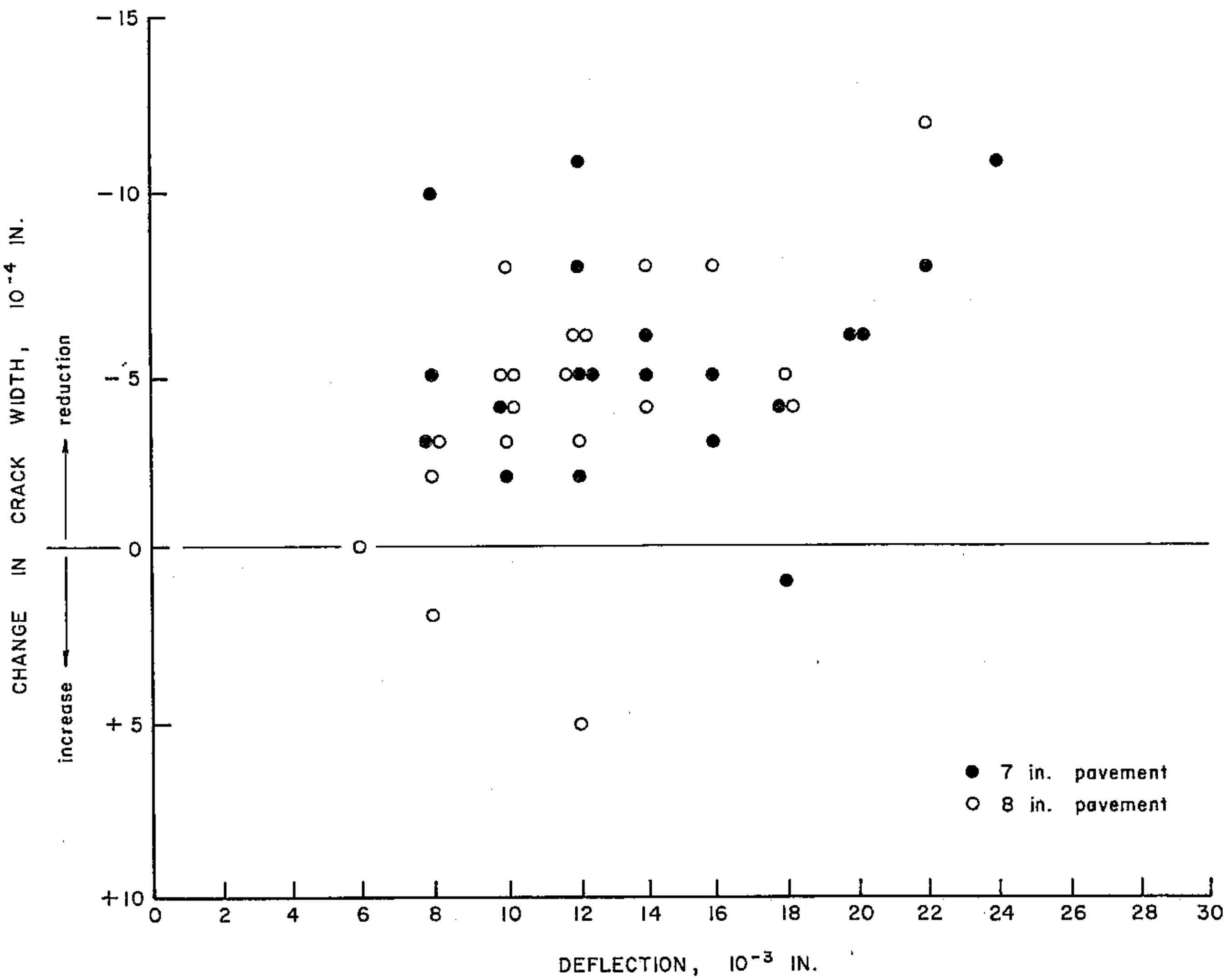
Crack Width

The crack width measurements were made with a Whittemore strain gage and reference plugs set in the pavements surface on each side of the cracks. In conjunction with this crack-width measurement, the static rebound deflection was taken when the load was on the crack. As expected, crack width decreased with load for 38 out of 41 locations. When the static rebound deflection and corresponding reduction in the crack width under load are plotted, as shown in Figure 8, the extent of dispersion of the points suggests that the reduction in the crack width at the pavement surface increases with the increase in deflection.

TRANSVERSE CRACKING

Transverse cracking and the factors affecting this cracking have been studied in considerable detail in the Illinois study. The ultimate crack pattern in a continuously reinforced concrete pavement results from shrinkage, temperature change, wheel load and climatical changes to which the pavement is subjected. The cracks that developed at an early age are generally the result of concrete shrinkage and temperature change.

The study has indicated that the slab thickness and type of steel reinforcement have little if any effect on transverse cracking. The depth of steel reinforcement



Note : 1 inch = 25.4mm

Figure 8. Relationship Between CRC Slab Deflection And Change In Crack Width At Pavement Surface, Unloaded To Loaded

below the pavement surface, seasonal time of slab construction (early spring or late fall), and the pavement age prior to traffic are the major variables that have an effect on transverse cracking. The depth of the reinforcement placement not only affects the number of transverse cracks that develop in the pavement, but also has a bearing on how tight the cracks are at the pavement surface and on the uniformity of the crack pattern.

Upon completion of each experimental project, an intensive crack survey was made at two 500-ft (152-m) locations selected randomly within a central portion of each mile of roadway. Generally, these surveys were repeated every two years. The average crack interval representing each crack survey was calculated and is reported under each variable discussion.

Depth of Steel Placement

The depth that the reinforcement is placed below the pavement surface has a major effect on the number of transverse cracks and tightness of transverse cracks at the pavement surface. Transverse cracking increases as the reinforcement is placed nearer the pavement surface; conversely, the average interval between transverse cracks decreases as the reinforcement is placed nearer to the pavement surface. The change in crack width from mid-winter to mid-summer increases as the depth of steel or crack spacing increases. Furthermore, the average crack interval decreases with increasing pavement age.

The change in average interval between transverse cracks due to depth of steel placement and age is summarized in Table 6. A definite relationship between average crack interval and depth of steel placement is obvious from this table. At all ages up to 6 years, the average crack interval is consistently higher for each incremental increase in steel-placement depth. The average crack interval increases or less cracks develop as the steel-placement depth is increased from 2 in. (51 mm)

TABLE 6

SUMMARY OF TRANSVERSE CRACK INTERVAL RELATIVE TO DEPTH OF STEEL

District	Test Site Identification	Depth of Steel (in.)*	Average Crack Interval (ft.) 1/ Initial	1 yr.	2 yr.	3 yr.	4 yr.	5 yr.	6 yr.
4	2a-CRC-7b2	2	9.0	2.8	-	-	-	2.6	-
	2a-CRC-7bM	Mid-depth	3.7	5.0	-	-	-	4.5	-
6	4a-CRC-7b2	2	4.4	3.4	2.9	2.7	2.6	2.5	2.4
	4b-CRC-7bM	Mid-depth	7.4	7.4	6.9	6.8	6.5	6.2	5.9
	4c-CRC-7f2	2	4.9	4.3	3.8	3.5	3.4	3.1	2.8
	4d-CRC-7fM	Mid-depth	6.0	5.7	5.6	5.5	5.3	5.1	4.8
	4e-CRC-8b2	2	5.0	3.5	2.6	2.2	2.2	2.1	2.0
	4f-CRC-8bM	Mid-depth	9.8	9.2	7.5	6.9	6.3	5.6	4.9
	4g-CRC-8f2	2	5.5	3.2	2.9	2.5	2.4	2.2	2.1
	4h-CRC-8fM	Mid-depth	9.8	8.5	7.1	6.5	6.0	5.5	4.8
2	1a-CRC-8f2	2	10.7	-	5.4	-	4.5	-	-
	1c-CRC-8fM	Mid-depth	8.7	-	5.2	-	4.2	-	-
7	5a-CRC-8b2	2	10.5	-	2.9	-	2.6	-	2.4
	5b-CRC-8b3	3	9.4	-	4.4	-	4.0	-	3.8
	5c-CRC-8bM	Mid-depth	10.8	-	5.9	-	5.5	-	5.4

*Note: 1 inch = 25.4 mm

1/ 1 ft. = 0.3048 m

to 3 in. (76 mm) or to mid-depth of slab. The initial crack-interval readings at a few locations are not following this general trend. The crack development within the first few weeks of slab life is very rapid. Therefore, variation of a few days at the time of initial crack survey could perhaps have caused this inconsistency.

Another special significant relationship shown by the data in Table 6 is between crack development and age. It is seen that for each test pavement the average crack interval decreased at a rather rapid rate for the first 1 or 2 years following construction, after which the rate of decrease slowed and remained very slow.

The depth of reinforcement placement not only affects the number of transverse cracks that develop in the slab, but also has major effects on the tightness of the cracks at the pavement surface. The change in the crack width increases as the depth of steel increases. The results of crack-width measurements taken in summer and winter on the research pavements are given in Table 7. The measurements were made with a Whittemore Strain Gage and reference plugs set in the pavement surface on each side of the cracks. The gage plugs were installed at the cracks which became apparent first because these cracks usually undergo the greatest amount of opening within their particular regions. The initial gage readings were taken during the afternoon of the same day of plug installation, at which time, owing to the increase in the pavement temperature, the cracks were likely to be in an almost closed condition. Concurrent with this reading, the existing width of the crack was measured with a scale microscope by peering down into the crack in line with the plugs to obtain the crack width near the surface. A change in the position of the microscope will give a different reading. Thus, it is not possible to accurately determine the actual crack width by this method. The change in width, however, is determined from the reference

TABLE 7

SUMMARY OF CRACK-WIDTH MEASUREMENTS
AT
PAVEMENT SURFACE

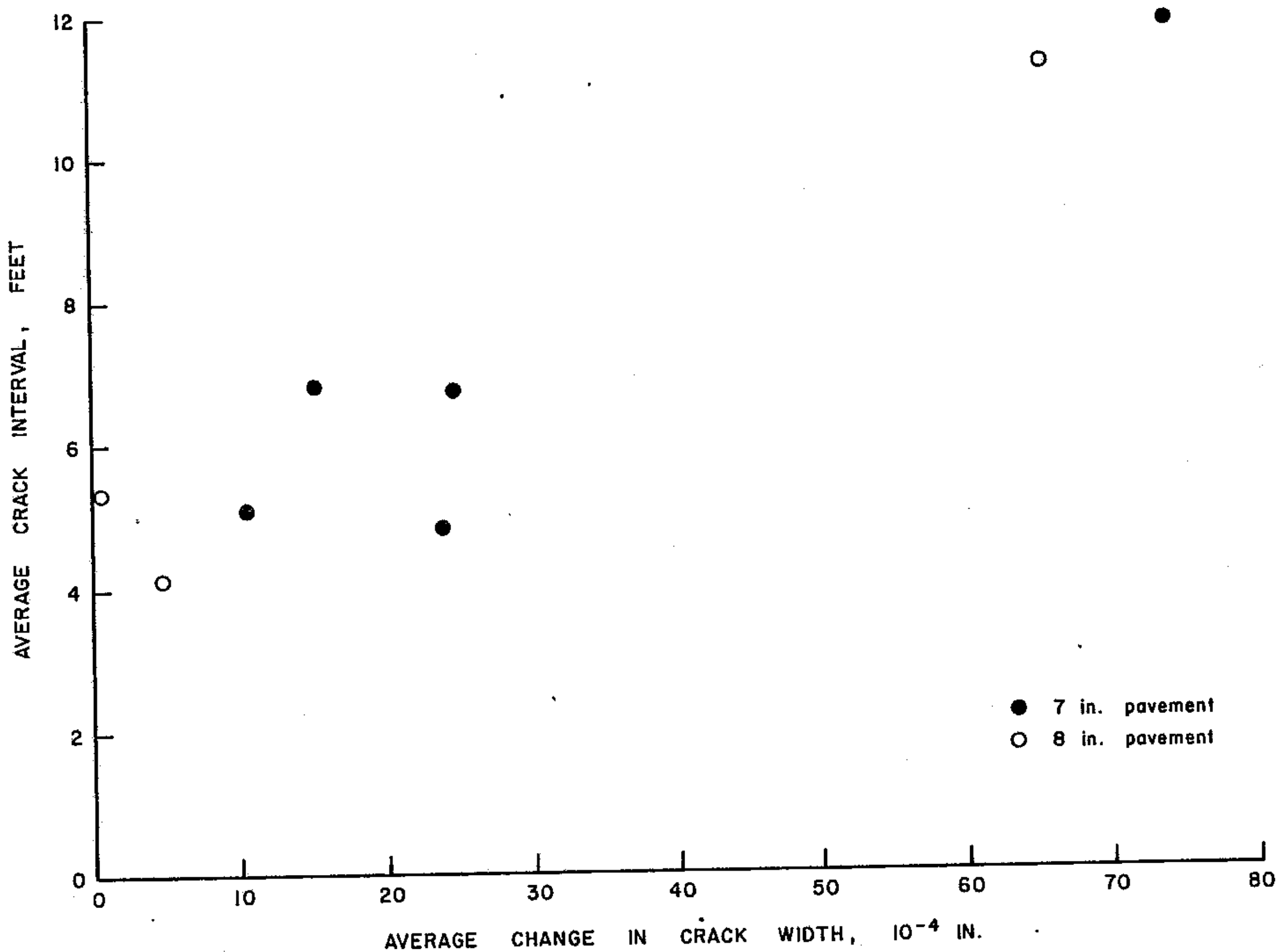
Depth of Reinforcement (in.)*	No. of Measurements 1969	No. of Measurements 1972	Average Crack Width (in.)* Winter	Summer	Change
2	16		.0184	.0183	.0001
3	4		.0253	.0241	.0012
Mid-depth	22		.0295	.0257	.0038
2		15	.0187	.0185	.0002
3		3	.0312	.0299	.0013
Mid-depth		22	.0327	.0288	.0039

*Note: 1 inch = 25.4 mm

plug measurements and can be duplicated. As shown in Table 7, the change in the crack width at the pavement surface is least when the steel is nearest to the surface. The crack width during mid-winter and mid-summer stayed almost the same for the pavement which had steel 2 in. (51 mm) below the surface. The change was small when the steel was placed at 3-in. (76-mm) depth, and somewhat greater when the steel was at mid-depth. Furthermore, it can be noted that there is no marked difference in the average change of crack width for 1969 and 1972 data.

The change in the crack width from mid-winter to mid-summer is not only effected by the depth of steel, but crack width increases as the crack spacing increases. As discussed before, the depth of steel affects the average crack spacing and change in the crack width at the surface; therefore, slabs with steel at mid-depth were selected and reported in Figure 9. There is no significant difference in the average crack interval of 7- and 8- in. (178- and 203-mm) slabs; therefore, data from both slab thicknesses are included in this graph. Both average crack interval and the average change in surface crack width seem to indicate a trend. This trend suggests that the average change in the crack width increases as the average crack interval increases.

The depth of the reinforcement placement also has a bearing on the uniformity of the crack pattern. The uniformity of the transverse crack pattern improves as the depth of steel increases. To illustrate variations in transverse crack patterns, the cracks in the selected lengths of pavement have been grouped into cells depending on the interval between individual cracks, and plotted as bar graphs. The information shown in Figure 10 is from Project No. 5. The slab is 8 in. (203 mm) thick and the depth of reinforcing steel is 2 in. (51 mm), 3 in. (76 mm), and mid-depth. There are only two projects in this study for which the slab thickness and depths of reinforcement are identical. Project 5 was selected randomly. For the sections with steel placed with 2 in. (51 mm) of cover,



Note : 1 inch = 25.4 mm
1 foot = 0.305 m

Figure 9. Influence Of Average Crack Interval On Average Change In Crack Width At Pavement Surface From Mid - Winter To Mid - Summer.

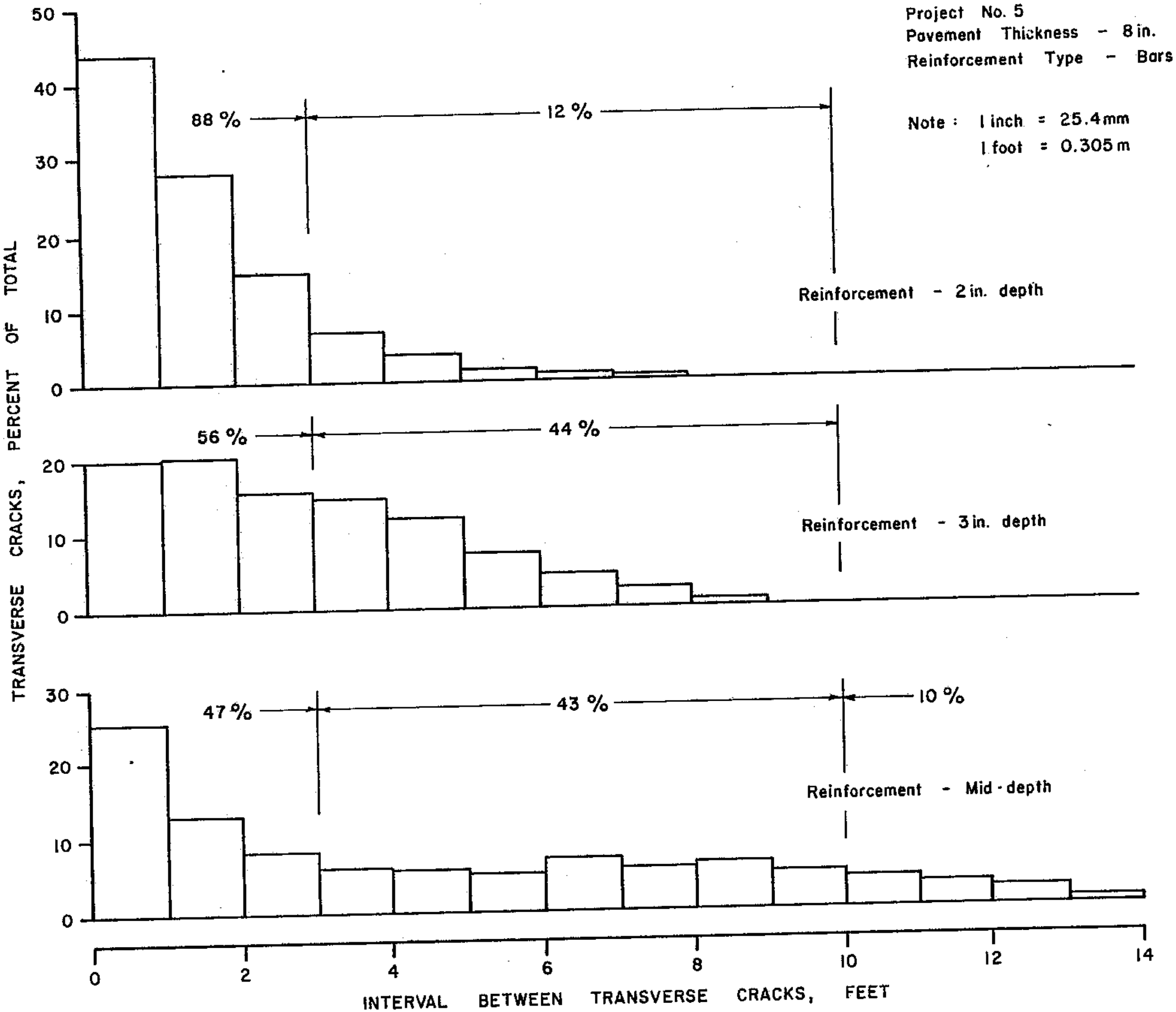


Figure 10. Crack Distribution For 8 inch Slab.

the average crack is 2.4 ft (0.73 m). Eighty-eight percent of the cracks are spaced less than 3 ft (0.91 m) apart, and the remaining 12 percent are within what is believed to be the desired range of 3 to 10 ft (0.91 to 3.05 m). When the steel is placed with 2 in. (51 mm) of cover, the crack pattern is not very uniform. A series of closely spaced cracks ranging from 6 in. (152 mm) to less than 3 ft (0.91 m) apart have occurred intermittently along the length of pavement. Several converging or "Y'ing" cracks have developed in the area of closely spaced cracks. For the sections with 3 in. (76 mm) below the pavement surface, the average crack interval is 3.8 ft (1.16 m). Fifty-six percent of the cracks are spaced less than 3 ft (0.91 m) apart, and the remaining 44 percent are within what is believed to be the desired range. At this depth, fewer cracks develop, the cracks still remain fairly tight at the surface, and the crack pattern is much more uniform with fewer areas of closely spaced cracks. When the steel is at mid-depth, the average crack interval is 5.4 ft (1.65 m), 47 percent of the cracks are less than 3 ft (0.91 m) apart, 43 percent are within 3 to 10 ft (0.91 to 3.95 m) apart, and 10 percent are greater than 10 ft (3.05 m) apart. When the steel is placed at mid-depth, the crack pattern is fairly uniform but many cracks appear to be more open at the surface.

Similar information was found from the 7-in. (178-mm) pavement. Again, a greater percentage of the cracks are spaced less than 3 ft (0.91 m) apart when the steel is 2 in. (51 mm) below the surface than when it is placed at mid-depth in the slab.

Slab Thickness and Type of Steel Reinforcement

Summary of transverse crack interval relative to slab thickness is shown in Table 8, and relative to the type of reinforcement is demonstrated in Table 9. The projects for which the information is reported in these tables were selected so that, as far as possible, only one variable was present and was evaluated.

TABLE 8

SUMMARY OF TRANSVERSE CRACK INTERVAL
RELATIVE TO SLAB THICKNESS

District	Test Site Identification	Slab Thickness (in.)*	Average Crack Interval (ft.) 1/						
			Initial	1 yr.	2 yr.	3 yr.	4 yr.	5 yr.	6 yr.
5	3a-CRC-7bM	7	15.6	-	12.5	-	12.2	-	11.9
	3b-CRC-8bM	8	17.0	-	12.0	-	11.4	-	11.3
6	4b-CRC-7bM	7	7.4	7.4	6.9	6.8	6.5	6.2	5.9
	4f-CRC-8bM	8	9.8	9.2	7.5	6.9	6.3	5.6	4.9
	4d-CRC-7fM	7	6.0	5.7	5.6	5.5	5.3	5.1	4.9
	4h-CRC-8fM	8	9.8	8.8	7.1	6.5	6.0	5.5	4.8

*Note: 1 inch - 25.4 mm
1/ 1 ft. = 0.3048 m

TABLE 9

SUMMARY OF TRANSVERSE CRACK INTERVAL
RELATIVE TO TYPE OF STEEL

District	Test Site Identification	Steel Type	Average Crack Interval, (ft.) 1/						
			Initial	1 yr.	2 yr.	3 yr.	4 yr.	5 yr.	6 yr.
4	2b-CRC-7bM	Bars	-	9.1	8.1	-	-	-	6.9
	2c-CRC-7fM	Fabric	-	9.2	7.5	-	-	-	5.2
6	4a-CRC-7b2	Bars	4.4	3.4	2.9	2.7	2.6	2.5	2.4
	4c-CRC-7f2	Fabric	4.9	4.3	3.8	3.5	3.4	3.1	2.8
	4b-CRC-7bM	Bars	7.4	7.4	6.9	6.8	6.5	6.2	5.9
	4d-CRC-7fM	Fabric	6.0	5.7	5.6	5.5	5.3	5.1	4.8
	4e-CRC-8b2	Bars	5.0	3.5	2.6	2.2	2.2	2.1	2.0
	4g-CRC-8f2	Fabric	5.5	3.2	2.9	2.5	2.4	2.2	2.1
	4f-CRC-8bM	Bars	9.8	9.2	7.5	6.9	6.3	5.6	4.9
	4h-CRC-8fM	Fabric	9.8	8.5	7.1	6.5	6.0	5.5	4.8

1/ 1 ft. = 0.3048 m

If a test pavement had more than one variable due to design, construction, or location, then the data were excluded. As shown in these tables, there is no significant difference in the average crack interval because of slab thickness or because of the type of reinforcement.

STRAIN OBSERVATIONS

This chapter describes the instrumentation and procedures used in an effort to expand the knowledge on stress levels and ranges in the steel and concrete of CRC pavements. Instrumentation was installed in eight test sections to determine strain and temperature measurements for a period which extended from the time of concrete curing to about eight years after construction.

Reinforcing steel is placed in CRC pavement to control the width of cracks caused by shrinkage and temperature variations of the concrete. By keeping the cracks tightly closed, the steel allows aggregate interlock to develop which provides load transfer across the cracks.

The strains in the steel and concrete at and near a crack are of special interest. To insure the development of a crack at a location compatible with the installed instrumentation, a transverse crack was induced in each test section by installing a two-inch-high metal strip on the subbase. This metal strip reduced the effective slab thickness by two inches, resulting in the formation of a transverse crack at the predetermined location.

Strain gages were mounted on the longitudinal steel reinforcement located at the crack and 6 in. (152 mm), 1 ft (0.30 m), 2 ft (0.61 m) and 4 ft (1.22 m) from the crack. It was hoped that adjacent cracks would be far enough away from the induced crack that the effect of the induced crack on the strains in the steel and concrete could be isolated.

Unfortunately, additional cracks developed within the instrumented panels at all locations except one (Figure 11). The extra cracks created more variables, making it difficult to evaluate the strains in the longitudinal reinforcement at the induced crack and at the intended intervals away from the crack. Consequently, most of the data presented in this report were collected from the one instrumented panel which was not affected by extra cracks. This panel consisted of 7-in. (178-mm) thick pavement with bar reinforcement located at mid-depth. The panel was located at Station 646 + 56 in the westbound pavement of Project 4 (Figure 1).

Instrumentation

To investigate the behavior of both the reinforcing steel and concrete in the vicinity of the induced crack, several types of instruments were installed near the center of each of the eight test sections (Figure 12). All instrumented panels were located within the length of one bar mat or sheet of fabric in the driving lane of the pavement (Figure 13).

The instrumentation consisted of electrical resistance foil strain gages installed on the longitudinal and transverse reinforcement bars, gages mounted on short pieces of steel bars to determine the effects of temperature, gages mounted on short Vycor (silica glass) rods in an attempt to provide a zero reference, concrete strainometers for measuring strains in the concrete, brass plugs mounted in the pavement surface for measuring strains at the surface of the pavement with a Whittemore gage, and thermocouples for measuring temperatures in the pavement at the level of the reinforcing steel.

All electrical resistance strain gages used were Bakelite-backed SR-4 foil gages. Procedures used for installing strain gages on reinforcement bars consisted of removing deformations from the reinforcing steel, applying epoxy, bonding the SR-4 gage to the steel, attaching lead wires, and waterproofing the installation. At each location, the leads from the gages were extended under the pavement to the

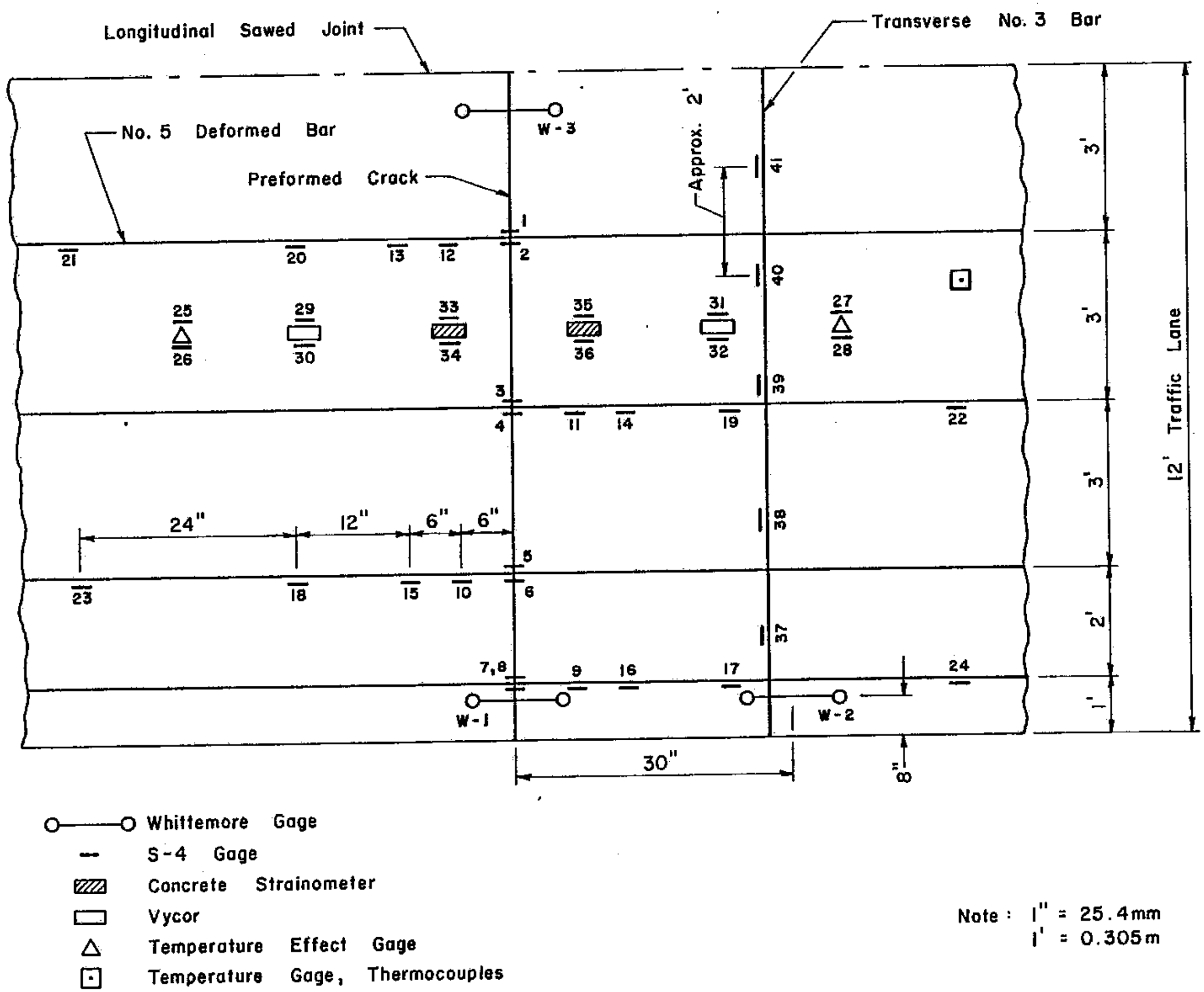


Figure 12. Arrangement Of Instruments In Pavement.

outer edge, and then extended in a common trench 9 to 12 in. (229 to 305 mm) below the earth subgrade across the shoulder area and 2 to 4 ft (0.61 to 1.22 m) down the side slope.

One of the most difficult applications of electrical resistance strain gages is the attempt to record strains over a period of several years with no opportunity to recheck the zero reading. The many factors which can influence the behavior of a strain gage have an opportunity to do so; moreover, over a long period of time the individual contribution to the error from each of the factors can become quite significant. The factors which can affect the indicated response of the gages include the type of adhesive used to mount the gage to the specimen, strain cycling, gage current, strain indicator temperature effects, and moisture and humidity.

In an attempt to account for the shift in zero reading with time, all strain gages mounted on steel were referenced to dummy gages mounted on Vycor (silica glass) rods. When readings were made, the Vycor gages were used as one arm of the Wheatstone bridge. It was expected that little change would take place in the gages mounted on Vycor since glass is not appreciably affected by temperature variation. However, when one Vycor-mounted gage was compared with another, significant changes with time and temperature were noted. This variation with time and temperature change was not consistent and no relationship with strain readings and temperature could be established. There was, however, much less variation with time in the gages mounted on Vycor than any of the gages attached to steel. Readings from the Vycor-mounted gages over a period of several years indicate that errors in the strain readings from all gages could be as much as $\pm$ 500 micro in. per in.

An attempt was made to measure the strain change in the reinforcing steel due to a change in slab temperature alone. At each instrumented panel, gages were mounted on opposite sides of two 2-in. (51-mm) pieces of reinforcing steel, and were embedded on each side of the preformed crack at mid-depth of the slab and between longitudinal reinforcing bars about seven ft (2.13 m) from the edge of the slab. As with all the steel-mounted gages, these gages were referenced to the Vycor-mounted gages by using the Vycor gages as dummies when readings were made.

Despite the care taken in installing the gages and the steps taken to try to obtain reliable readings over a long period of time, most of the gages began to yield erratic and unreliable results after about six months to a year.

Properties of Reinforcement Bars

The longitudinal reinforcement in each lane for 7-in. (178-mm) CRC consisted of 19 No. 5 bars of 60,000 psi (414,000 kPa) grade steel. The bars comprised 0.6 percent of the cross-sectional area of the pavement. Based on the results of 23 tests, the average yield strength was 77,463 psi (534,700 kPa) and tensile strength was 133,260 psi (919,000 kPa). If a modulus of elasticity of 30 x 10^6 psi (207 x 10^6 kPa) is assumed, then the steel used in this pavement will begin to yield at about 2600 micro-in./in. The strain for the tensile strength of 133,260 psi (919,000 kPa) is about 4,450 micro-in./in.

The transverse bars consisted of No. 3 bars of intermediate grade steel spaced at 25 in. center to center. They have a yield strength of 53,700 psi (370,500 kPa) and a tensile strength of 80,600 psi (556,000 kPa).

Strain in Steel During Concrete Curing

Plots of the strains measured in the longitudinal reinforcement bars during the curing of the concrete until cracking occurred are given in Figures 14 and 15. Figure 14 shows the plot of three pairs of gages located at the crack. Each plotted line represents the average of the two gages bonded to opposite sides of reinforcement bars located at different distances from the

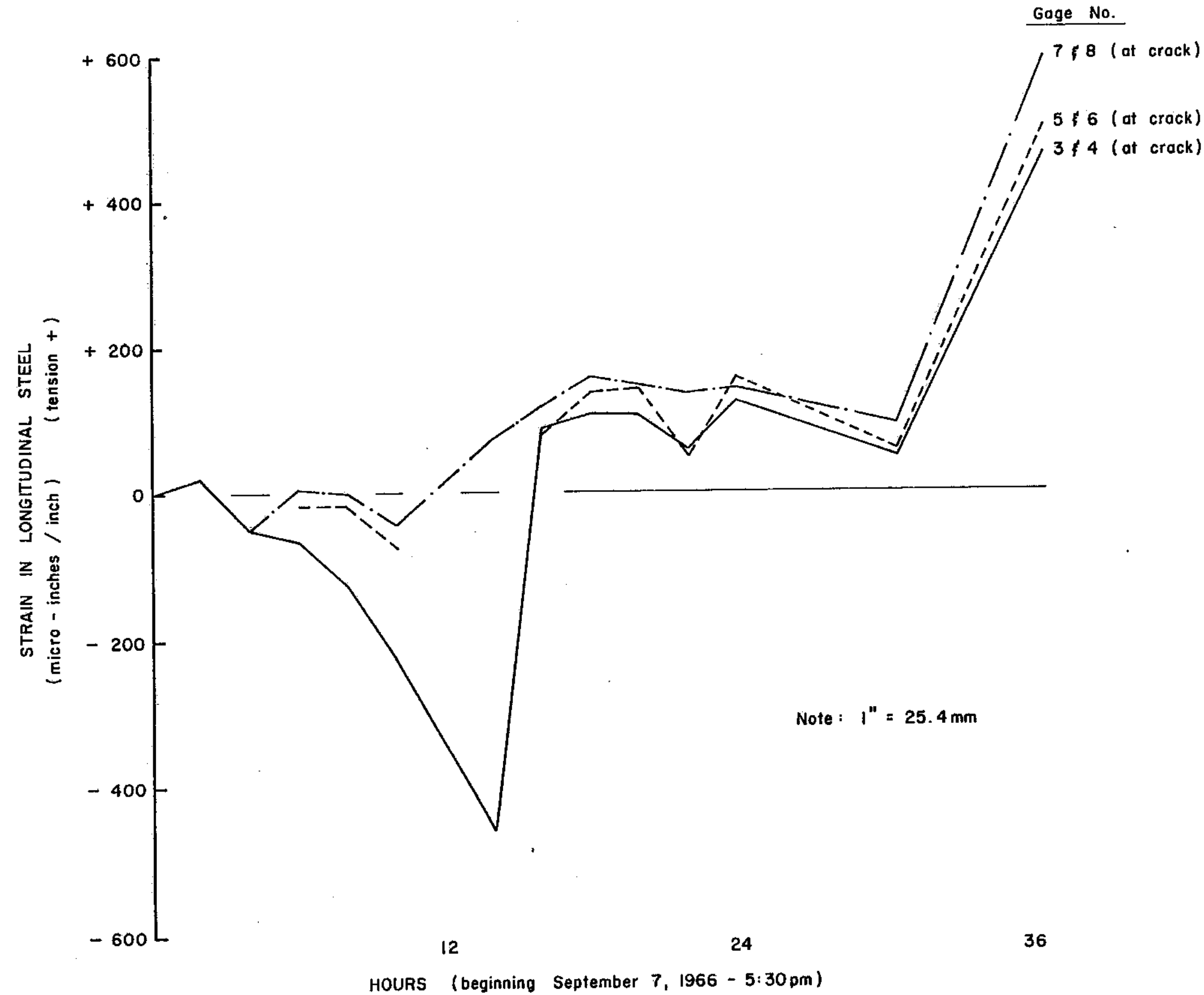


Figure 14. Strains In Longitudinal Reinforcement At Crack During Period From Concrete Placement To Formation Of Crack.

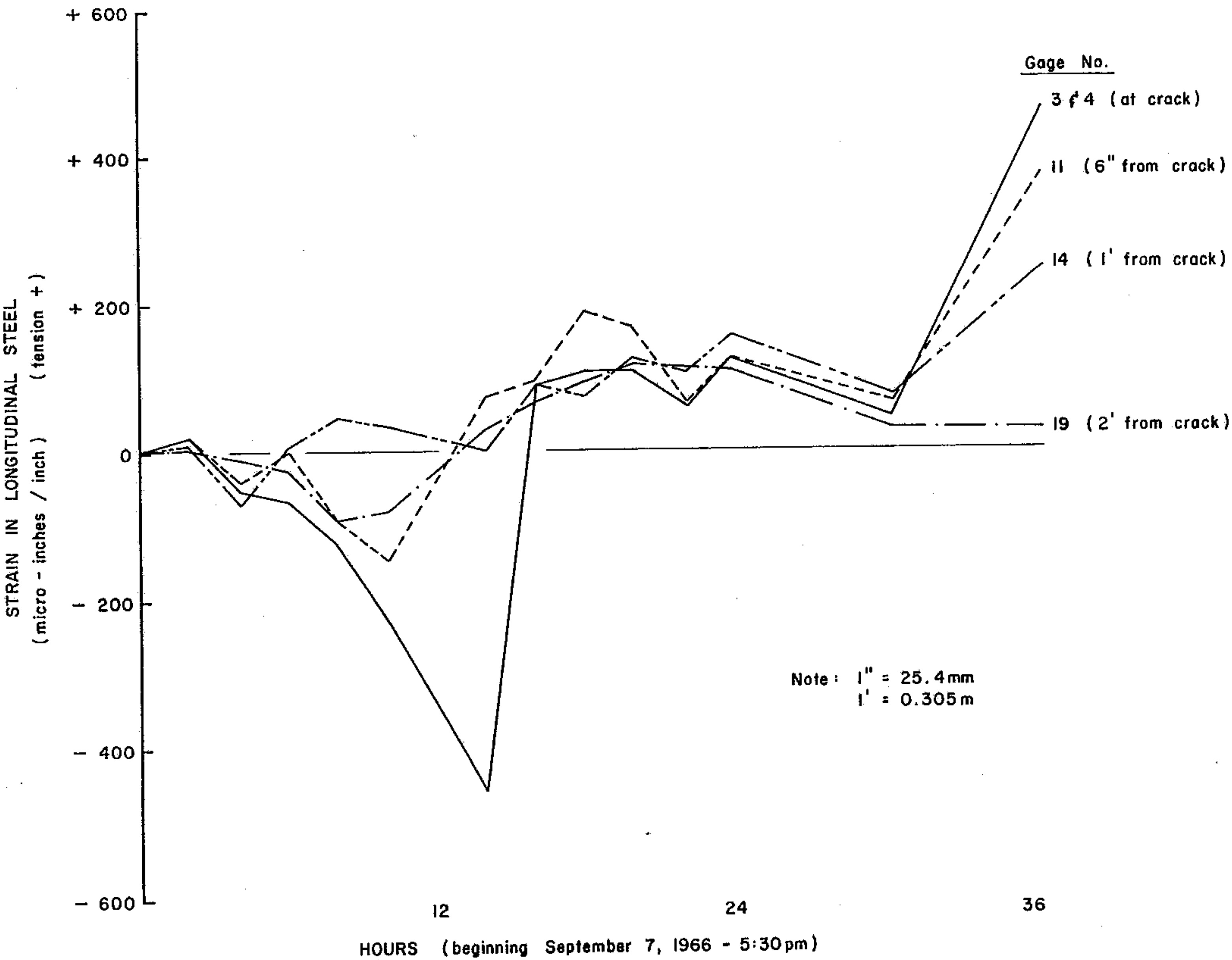


Figure 15. Strain Along Longitudinal Reinforcement Bar During Period From Concrete Placement To Formation Of Crack.

edge of the pavement (Figure 12). The plot indicates that, after a very small initial tensile strain possibly caused by heat of hydration during the early curing stage, the strains changed to compression during about the first 12 hours. The compressive strains may be caused by shrinkage of the concrete during this period. The reason for the larger compressive strain indicated at gages 3 and 4 is not known but may be due to measurement errors. Between 12 and 15 hours after readings began, all measurements indicated that the steel was in tension and remained fairly constant until about 30 hours had passed. Some time between 30 and 36 hours, the induced crack formed and the tension in the steel increased rapidly to values of about 450 to 600 micro-in./in., with the tensile strain being highest in the bar nearest the edge of the pavement and diminishing with distance from the edge.

Gages 5 and 6 ceased functioning shortly after this period. Gages 1 and 2 are not plotted because they did not function from the beginning.

Figure 15 shows the strains during the first 36 hours at different locations on the same reinforcement bar. Gages 3 and 4 are located at the crack, Gage 11 is located 6 in. (152 mm) from the crack while Gages 14 and 19 are located 1 ft and 2 ft (0.30 m and 0.61 m) from the crack, respectively. The indicated strains are essentially compressive for the first 12 to 15 hours, after which all gages begin to indicate tensile strains.

At 36 hours, shortly after the induced crack formed, the tensile strain was increasing and was highest at the crack and diminished with distance from the crack.

Strain in Steel During First 18 Months

The strains in the longitudinal steel at Gages 3 and 4, and Gages 7 and 8 during the first 18 months after cracking are shown in Figure 16. During

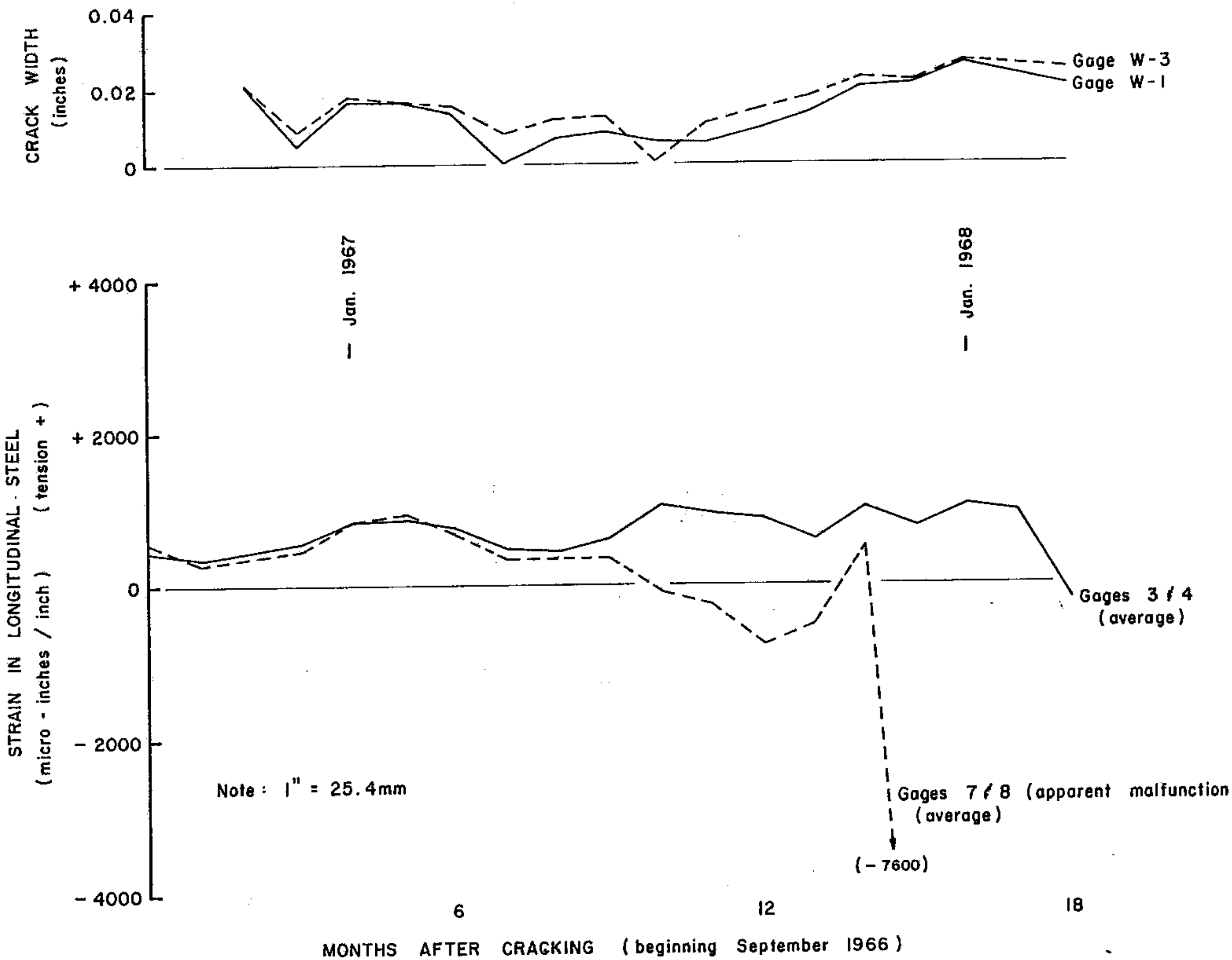


Figure 16. Strains In Longitudinal Reinforcement At Crack And Crack Width Versus Time.

the first nine months the two pairs of gages indicate similar strain patterns, with both sets of gages remaining in tension throughout this period. The maximum strains indicated by both pairs of gages were just under 1,000 micro-in./in., which would represent a steel stress of about 30,000 psi (207,000 kPa). After nine months the indicated strains at the two locations began to diverge, with Gages 3 and 4 remaining in tension while Gages 7 and 8 moved toward compression. At 14 months, Gages 7 and 8 indicated a sharp drop to about 7,600 micro-in./in. compression, while Gages 3 and 4 remained in tension. From this point on, Gages 7 and 8 provided unreliable readings. Gages 3 and 4, and 7 and 8 were positioned at the location of the induced crack. Figure 16 also shows the variation in crack width for the same time period, as measured by Whittemore gages. The measurements indicate that the crack width varied between 0 and about 0.025 in. (0.635 mm) during this period. The maximum openings generally occurred during the winter, as would be expected.

Long-Term Strain in Longitudinal Steel

Figure 17 is a plot of the strains in Gages 3 and 4 located at the induced crack over a period of seven years after the induced crack formed. Gages 3 and 4 were the only pair of gages at the crack remaining operable after 14 months. During the first 18 months, Gages 3 and 4 remained in tension, as described in the preceding section. After the first 1 1/2 years, Gages 3 and 4 moved toward compression and after 2 years and nine months reached compressive strains of about 2,800 micro-in./in., which would exceed the proportional limit of the reinforcement bar in compression. The maximum indicated tensile strain of about 2,700 micro-in./in. occurred about 4 1/2 years after cracking. The general long-term trend of the curve is toward compression, except for the shift toward tension between 4 and 4 1/2 years, after which the curve resumes the move toward compression.

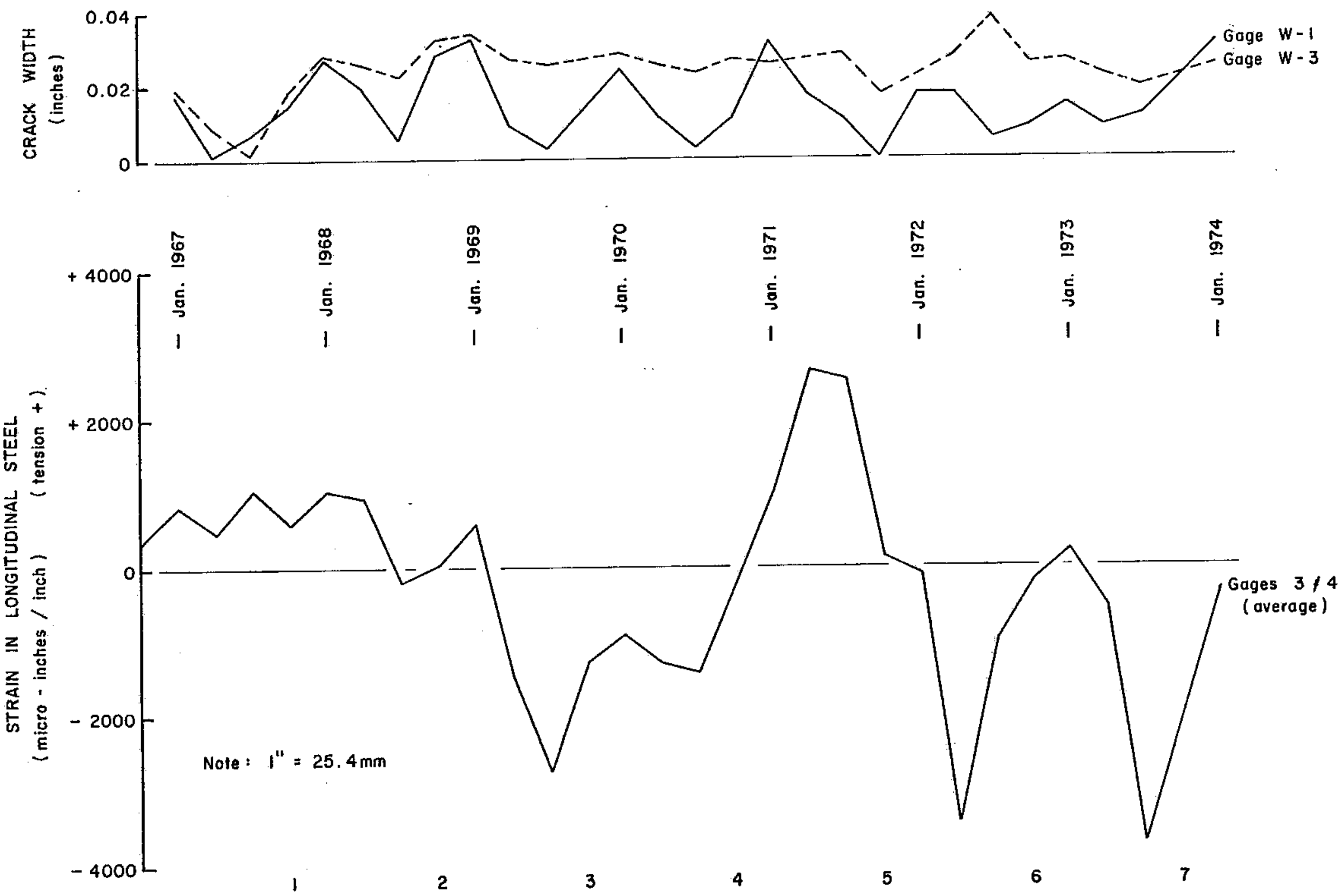


Figure 17. Long – Term Strain In Longitudinal Reinforcement And Long Term Variation In Crack Width.

Figure 17 also shows a plot of the variation in crack width for the seven-year period after cracking. Gage W-1 is located near the edge of the pavement, while Gage W-3 is located near the centerline of the pavement. It may be noted that Gage W-1 indicated that the crack opened and closed each year, with the maximum opening being about 0.027 in., (0.686 mm). Gage W-3, near the center of the pavement, indicated that the crack opened up and tended to stay open during the seven-year period.

The reinforcement bar to which Gages 3 and 4 were attached was located about midway between Whittemore gages W-1 and W-3. In general, the peaks of the strain curve coincide with the peaks of the crack width curve and, except for 1971, the peaks occurred in the winter, as would be expected.

Figure 18 is a plot of the strains occurring in one reinforcement bar at varying distances from the induced crack. Gages 3 and 4 are located at the crack, while Gage 11 and Gage 14 are located 6 in., (152 mm) and 1 ft (0.305 m) from the crack, respectively. Measurements from Gages 19 and 22, located 2 ft (0.61 m) and 4 ft (1.22 m) from the crack, are not plotted in Figure 17 to reduce the confusing effect of too many lines.

It would be reasonable to assume that the strains would diminish with distance from the crack and would be of the same sign as the strains at the crack. The plotted measurements in Figure 18 do not always indicate this. The magnitude of the strains away from the crack often exceed the magnitude of strains at the crack and are sometimes of opposite sign. This discrepancy between expected behavior and observed values of strain casts doubt on the validity of the measured strains. One trend evident from all three plotted lines is the gradual shift from tension toward compression with time. It is uncertain whether this shift is real or the result of drift in the instrumentation with time.

Note : 1" = 25.4mm

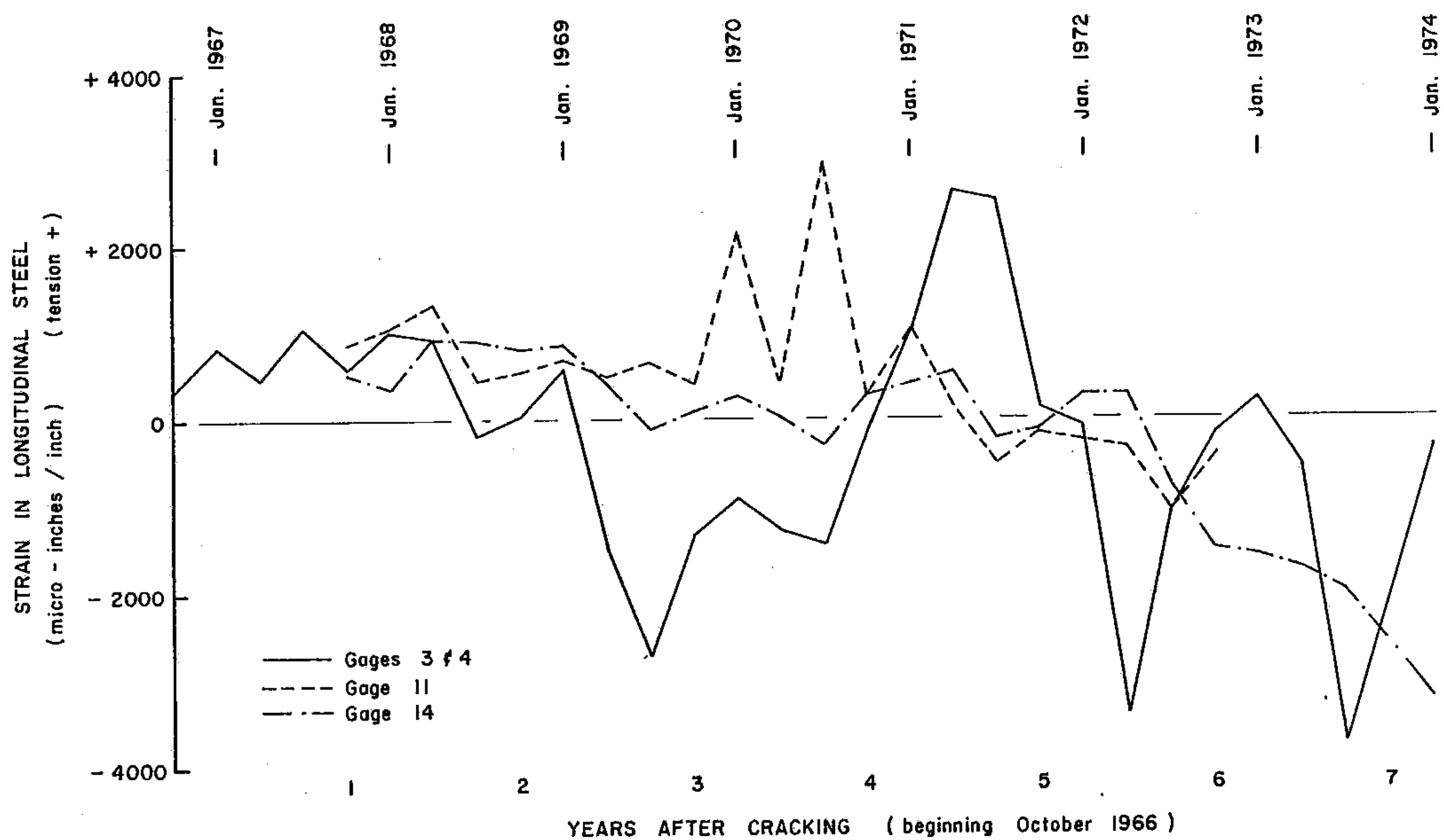


Figure 18. Long - Term Strain Along Longitudinal Reinforcement Bar.

Long-Term Strain in Transverse Steel

Five gages (Nos. 37 through 41) were installed about 2 ft (0.61 m) apart along the length of one transverse reinforcement bar in the test section. Of the five gages, only Gages 38 and 41 yielded readings which appeared to be within reason. A plot of these readings is shown in Figure 19 for the period from January 1968 to October 1971. Immediately before and after this period no readings were obtained from these gages. The measurements indicate more moderate changes in strain than measured in the longitudinal steel. The gradual downward trend from tension to compression is also evident in these plots, as well as in the plots of the longitudinal steel.

Strain in the Concrete

An attempt was made to measure the strain in the concrete by installing concrete strainometer devices (Figure 20) at 1 ft (0.305 m) on either side of the induced cracks at each test section. Two SR-4 strain gages were cemented to the strainometer body in such a way as to allow one gage to measure the longitudinal strains and the second gage mounted transversely to achieve temperature compensation. These two gages were wired to form two arms of a Wheatstone bridge.

When the induced crack appeared, the gages on the concrete strainometers at most locations were showing about 100 micro-in./in. of tension, which is equivalent to a stress of about 300 psi (2070 kPa), assuming a modulus of elasticity for the concrete of 3×10^6 psi (20.7×10^6 kPa). The tensile strain increased to about 160 micro in./in. during the first winter after paving. The gages on the strainometers began to give erratic results during the first spring thaw, and by June 1967 almost all gages produced unreliable data.

Note : 1" = 25.4 mm

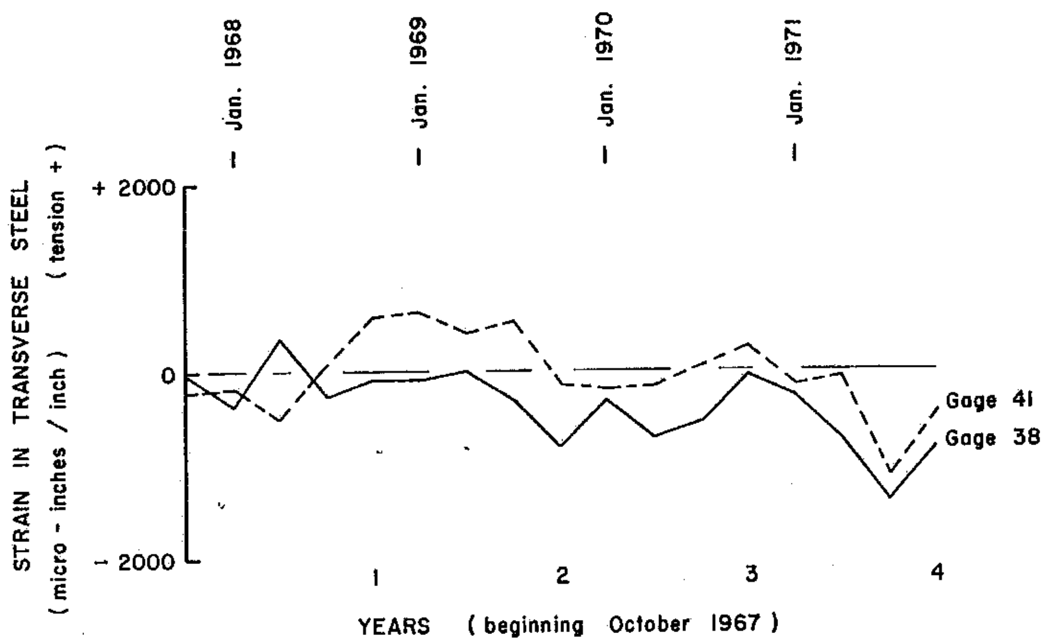
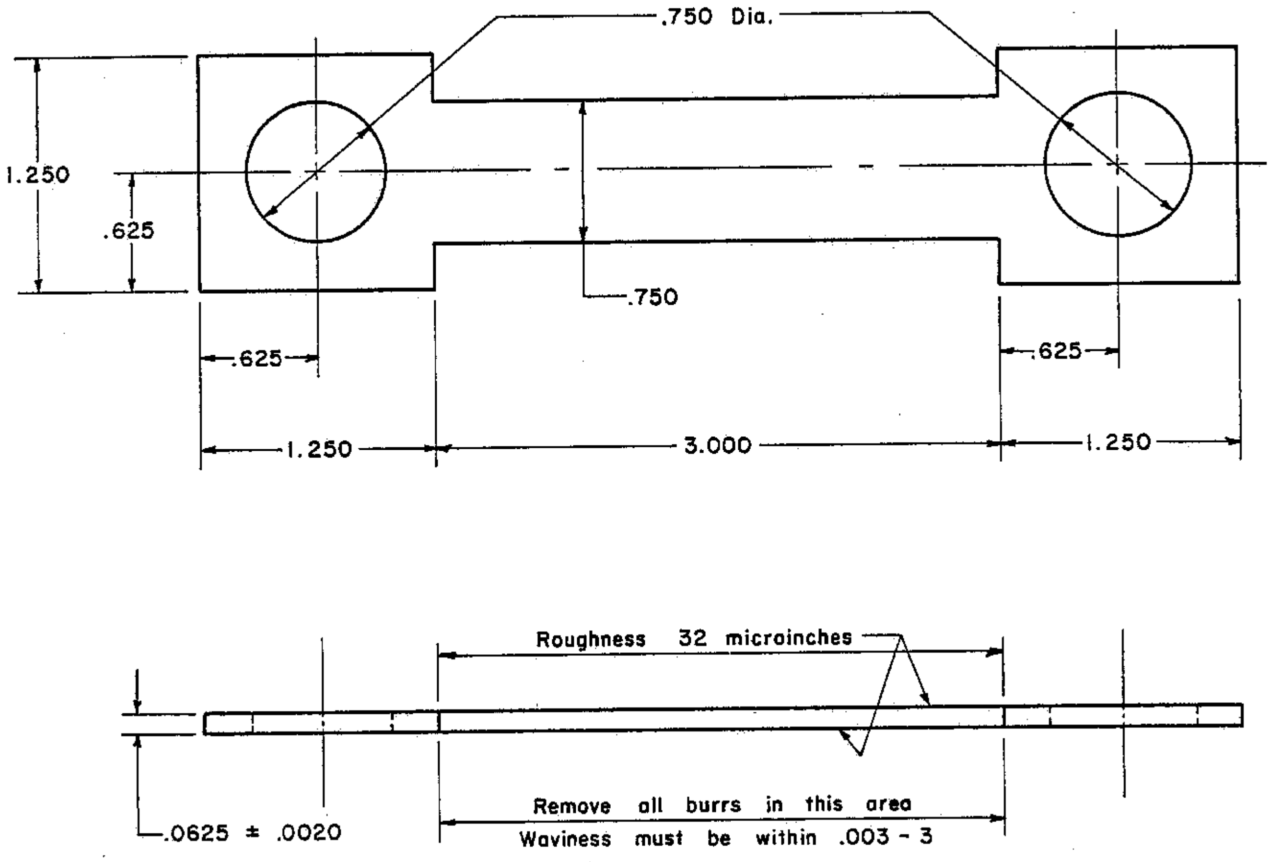


Figure 19. Long - Term Strain In Transverse Reinforcement Bar.



Notes : (1) Drill all holes before finishing.

(2) General tolerance ±.007 unless otherwise specified.

(3) All dimensions are in inches, 1 in. = 25.4 mm.

(4) Material - Mild steel, maximum carbon 0.15 percent.

Figure 20. Strainometer Body.

PERFORMANCE

In general, the performance of Illinois CRC pavements to date appeared to be quite satisfactory. At a few places where excessive crack widths have been found, localized pavement failures have been traced to construction deficiencies consisting of either a total absence of longitudinal steel or insufficient lap. Pavement failures also have been associated with inadequate consolidation of the concrete at construction joints. Signs usually show up shortly after paving, and distress occurs during the first winter. Practically all failures developed before the contracts were finalized, and were repaired by the contractors at their expense. Most of these failures occurred while both the engineers and contractors were gaining experience in constructing CRC pavements. As a result of these experiences, the specifications now require all lapped reinforcement sheets to be securely tied both transversely and longitudinally, regardless of the method of placement. The specifications were further revised to call for hand vibration to extend for 10 ft on each side of the construction joint for the full width of the pavement (1).

Edge pumping was the second distress manifestation observed (10, 11). This distress manifestation was observed on three construction projects two to three years after the pavement was opened to traffic. This distress mainly was limited to the 7-in. (178-mm) slab on a 4-in. (102-mm) trenched granular subbase. Light pumping was observed along the outer edges, and also the transverse cracks in the traffic lanes appeared somewhat wider at the surface than is normal. Also, advanced pavement distress was observed at several isolated locations. The pumping did not appear to be associated with any particular characteristics of the road profile, but was generally scattered along a major portion of the pavement length. The sign of pumping existed as small deposits of granular material along the joint between the pavement edge and paved shoulder. During field studies, the pavement edge at three locations, where most severe pumping was observed, was exposed to check for voids under the slab. At each location voids (5 to 8 in. (127 to 203 mm) long, 1/2 to 1 1/2 in. (13 to 38 mm) deep and extending under the slab only a few inches) were found. A smaller cavity sufficient for a wire or steel tape to be inserted extended further under the slab. The edge deflections, measured by the Benkelman beam under 18,000-lb (8165 kg) rear-axle load, were greatest where evidence of pumping was most pronounced. The deflection values were somewhat greater (0.035 in.) (0.89 mm) in the traffic lanes than in the passing lanes (0.029 in.) (0.74 mm). It was concluded from the results of the study that the pumping and wider cracks in the traffic lanes were associated with free water entering principally at the pavement edges and collecting in the 4-in. (102-mm) trenched granular subbase. The subbase was acting as a reservoir, retaining excessive amounts of water for extended periods of time and creating periods of weakened subgrade support. During these periods, heavy loads caused greater deflection of the 7-in. (178 mm)-thick slab and pumping of the subbase material by scouring action. No major pumping problem was found when the slab was 8-in. (203-mm) thick. To prevent or delay severe pavement distress, which was imminent, from developing early in the service life of the pavements, a positive underdrain system along the outside shoulders immediately adjacent to the pavement edge was recommended and installed. Further, it was recommended that the joint between the pavement edge and paved shoulder be sealed and maintained in a sealed condition. These corrective treatments appear to have somewhat alleviated these problems. After about 7 years of service, the severity of structural distress at a few isolated locations was such that full-depth CRC patches were required. Generally, distress progressed gradually from the outer edge or wheelpath, and consisted of open crack or cracks indicative of steel rupture, punchout and disintegration of concrete. Pumping and spalling developed rapidly after the cracks opened. To alleviate surface roughness, bituminous maintenance was used at many locations. These types of failures could be associated with the questionable structural adequacy of a given pavement, subbase, or subgrade when exposed to heavy loading. Spalling along the majority of the transverse cracks was observed. This spalling was minor on the 8-in. (203-mm)-thick slab but was severe on the 7-in. (178-mm)-thick slab. Severity of the spalling phenomenon suggests that thin slabs are apparently experiencing excessive deflections. There is a strong indication that spalling may be attributed to the number of wheel load repetitions, although possibly other parameters establish the condition that allows the spalling to occur (12). A variable amount and kind of distress in the concrete was observed at some isolated locations. The majority of the concrete distress consisted of "D" cracking. This type of distress was progressive in nature, and at a few locations ended in total disintegration of the concrete. Other research has indicated that this type of distress is associated with the physical properties of the coarse aggregate in the concrete mix (13). Illinois is currently conducting laboratory and field tests to identify the magnitude of the problem, and to determine what course of action should be taken to correct the problem.

The traffic that the experimental pavements has carried is typical in volume and character of the traffic on many primary highways servicing a high percentage of heavy-truck traffic. Most of the experimental pavements consist of two lanes each way on a divided highway. The outer lane carries the greater number of heavy trucks. This uneven distribution of heavy traffic is the main cause, among other causes, that most of the failures took place in the driving lanes. The greater volume of traffic using the outer lane causes considerably more raveling, spalling, popouts and wearing of the edges of the transverse cracks than the lighter traffic on the passing lane. The effect of variable traffic load conditions on the amount of patching for each research pavement as of fall 1977 is presented in Table 10. That table contains the total amount of patching, including those patches due to construction deficiencies. The amount of patching is related to the slab thickness, depth of reinforcement below the pavement surface and cumulative loadings. The type of reinforcement has little, if any, effect on the amount of patching.

The patching decreases as the slab thickness increases. Projects in District 5 and 6 are the only pavements in this research study where the slab thickness variable was incorporated during construction and can be evaluated. As can be seen from the District 5 data, the amount of patching for 8-in. (203-mm) CRC pavement is nil and for the 7-in. (178-mm) CRC slab is 6.5 sq ft(.60 sq m). Similar information is shown for the District 6 project, but the amount of patching is relatively greater than on any other project in this research study. Many of the earlier failures took place in the 7-in. (178 mm) CRC westbound pavement on both sides of a railroad grade separation structure and were found to be associated with the construction deficiencies and very poor subgrade drainage. The 8-in. (203-mm) CRC eastbound pavement immediately opposite this location has been performing satisfactorily. Again, the amount of patching decreased as the slab thickness increased from 7 to 8 in. (178 to 203 mm).

The second finding interpreted from this table is the relationship between the amount of patching and the depth of reinforcement below the pavement surface. As can be seen, the amount of patching increases as the depth of reinforcement increases.

The third finding is the relationship between the amount of patching and the cumulative 18-kip (8172-kg) single-axle equivalent loadings. The amount of patching

TABLE 10

CUMULATIVE TRAFFIC AND PATCHING ON EXPERIMENTAL PAVEMENTS

Dist.	Year Opened	Slab Thickness (in.)*	Reinforcement Type	Reinforcement Depth (in.)*	Cumulative 18-Kip ESAL	Patching Sq. Ft. Per 1000 Sq. Ft.	Aggregate Type	14 Days Flex. Strength PSI
2	1964-65	8 CRC	Fabric	2	984,000	1.2	Limestone	910
	"	"	"	3	"	4.2	"	"
	"	"	"	4	"	0	"	"
	"	10 PCC	-	Mid-Depth -	-	2.6	"	-
4	1964	7 CRC	Bars	2	2,454,000	0.9	Gravel	917
	"	"	"	3.5	"	6.7	"	"
	"	"	Fabric	Mid-Depth 3.5	"	9.3	"	"
	"	10 PCC	-	-	"	0.9	"	"
5	1965	7 CRC	Bars	3.5	2,598,000	6.5	Limestone	754
	"	8 CRC	"	4	"	0	"	"
	"	10 PCC	-	-	"	0	"	"
6	1966	7 CRC	Bars	2	1,312,000	49.4	Limestone	701
	"	"	"	3.5	"	52.7	"	"
	"	"	Fabric	2	"	44.6	"	"
	"	"	"	3.5	"	38.5	"	"
	"	8 CRC	Bars	2	"	7.8	"	"
	"	"	"	4	"	0	"	"
	"	"	Fabric	2	"	0	"	"
	"	"	"	4	"	0	"	"
7	1963	8 CRC	Bars	2	6,165,000	7.0	Limestone	807
	"	"	"	3	"	12.7	"	"
	"	"	"	4	"	30.9	"	"
	"	10 PCC	-	-	"	0	"	"
9	1965	7 CRC	Fabric	2	1,086,000	0	Limestone	828
	"	"	"	3.5	"	4.4	Gravel	"
	"	10 PCC	-	-	"	0	-	"

* Note: 1 inch = 25.4 mm PSI = 6.895 KPa

increases as the cumulative loading increases. This relationship exists for all 7- and 8-in. (178- and 203-mm) pavements, except for the 7-in. (178-mm) pavement in District 6 where many premature failures, as previously explained, took place. Except for the 7-in. (178-mm) CRC pavement in District 6, all other pavements are serving adequately for the intended loading.

The pavement in District 7, which is 8-in. (203-mm) CRC and was opened to traffic in 1963, has lately required a relatively higher level of maintenance. In spite of the extensive patching that has been performed, the continued deterioration of the pavement at an increasing rate is expected in the future. In terms of the number of years of service life normally expected, this pavement appears to have experienced a greater number of premature failures. This 8-in. (203-mm) CRC pavement already has served more than 6 million equivalent 18-kip (8172-kg) single-axle load applications. Based on these load applications, the present design would have called for a thicker pavement. Thus, it is apparent that this pavement was underdesigned.

The primary reason of the underdesign was the lack of an adequate knowledge of the pavement design-performance relationship. This relationship has been derived from the AASHO Road Test data even though continuously reinforced concrete pavement type was not included at the test site. The ratio of the thickness of CRC pavement (0.6 percent longitudinal steel) to the thickness of standard reinforced jointed pavement for equivalent performance was determined to be 0.7. This ratio was developed through an analysis of the data available from the Vandalia experimental pavements (14). These were the only actual performance data on CRC pavement that were available when this research study was undertaken. Observing and comparing the behavior of pavements (7-in. (178-mm) CRC, 8-in. (203-mm) CRC and 10-in. (254-mm) PCC) constructed under the present research study led to the conclusion that the 0.7 ratio that originally was assumed based on the small amount of information then available was too low. In addition, the difficulties with the behavior of the 7-in. (178-mm) CRC pavements on 3 projects have led to the conclusion that a 7-in. (178-mm) thickness of CRC pavement may not have the structural characteristics required to overcome the variety of construction imperfections and other deficiencies that sometimes contribute as a group to the failure of pavements carrying heavy traffic. Also, findings have reemphasized the importance of providing subbases and subgrades which are not susceptible to densification or loss of support under repeated loadings. Based on these findings, the use of a 7-in. (178-mm) CRC slab thickness and nonstabilized subbase has been discontinued in Illinois. The Illinois design ratio of thickness of continuously reinforced concrete pavement to standard reinforced concrete pavement has been changed from 0.7 to 0.8, and 8 in. (203 mm) will be considered the minimum thickness for CRC pavement on primary highways. The pavement will be placed over a 4-in. (102-mm)-thick stabilized aggregate subbase and will have a positive underdrain system.

Control sections of standard construction (10-in. (254-mm) PCC) were included in all projects except District 6. Service behavior studies were conducted through the years and these control pavements performed adequately and are generally giving good service. The defects that have occurred have been for the most part associated with transverse joints.

RIDING QUALITY

The principal aim of a highway is to serve the traveling public. A pavement that provides a smooth ride and maintains a smoother ride over a longer period is presumed to be serving better than one which offers a rough ride, other factors being equal. The surface smoothness of Illinois research pavements is being measured with a BPR-type roadometer. Roughness measurements made in 1966 through 1973 on the experimental pavements constructed from 1963 to 1966 are given in Table 11. Each value of the Roughness Index shown is the average for measurements made in each of the four wheelpaths of the 24-ft (7.32-m) pavement. There is no trend which can be associated with the type of pavement and other variables within the continuously reinforced concrete pavement study.

SUMMARY OF PRINCIPAL FINDINGS

Observation of the behavior of the Illinois experimental CRC pavements and adjacent conventional PCC pavements indicate the following:

1. Continuously reinforced concrete pavements (7-in. and 8-in.) (178-mm and 203-mm) deflected greater than the edge of conventional 10-in. (254-mm) PCC pavements.
2. There is no pronounced trend or difference in deflections due to the type of reinforcement, depth of reinforcement, and age of the pavement.
3. The depth of reinforcement not only affects the number of transverse cracks that develop in the pavement, but also has a bearing on how tight the cracks are at the pavement surface and on the uniformity of the crack pattern.
4. The slab thickness and type of steel reinforcement have little, if any, effect on transverse cracking.
5. After the placement of the concrete, the heat of hydration may have caused a slight initial tension in the reinforcing steel and then shrinkage of the concrete and bond development become the dominating factors. At the time of crack development, an abrupt change from compression to tension or from very low tension to a relatively higher level of tension took place in the reinforcing steel.

TABLE 11

ROUGHNESS INDEXES FOR EXPERIMENTAL PAVEMENTS AND ADJOINING CONVENTIONAL PAVEMENTS

District	Pavement Thickness (in.)*	Steel Type	Steel Depth (in.)*	Roughness Index 2/ (in. per Mile) 1/ 1966	1968	1969	1970	1973	1974
2	8	Fabric	2		88		105	101	
	8	Fabric	3		84		105	104	
	8	Fabric	4		88		109	106	
	10				93		120	113	
4	7	Bars	2		54	51		71	
	7	Bars	3.5		80	81		98	
	7	Fabric	3.5		82	78		92	
	10				70	66		80	
5	7	Bars	3.5		81	74**		85**	
	8	Bars	4		77	72**		103**	
	10				75	73		77	
6	8	Fabric	4		100		128	115	106
	8	Fabric	2		86		109	98	96
	8	Fabric	4		109		134	133	128
	8	Bars	2		85		118	109	99
	8	Bars	4		93		115	112	103
	8	Bars	2		92		134	124	110
	7	Fabric	2		84		106	111	106
	7	Fabric	3.5		83		104	111	108
	7	Fabric	2		88		116	116	116
	7	Bars	3.5		106		136	137	138
	7	Bars	2		85		120	115	106
	7	Bars	3.5		92		125	118	113
7	8	Bars	2		84	74		85	
	8	Bars	3		84	70		86	
	8	Bars	4		92	89		106	
	10				98	88		99	

TABLE 11 (continued)

ROUGHNESS INDEXES FOR EXPERIMENTAL PAVEMENTS AND ADJOINING CONVENTIONAL PAVEMENTS

District	Pavement Thickness (in.)*	Steel Type	Steel Depth (in.)*	1966	1968	1969	1970	1973	1974
9	7	Fabric	2	82	85		95	89	
	7	Fabric	3.5	80	79		88	83	
	10			84	86		102	92	

Note: *1 inch ≑ 25.4 mm

** Outside Lanes only

1/ Inch per mile = 15.8 mm per km

2/ Illinois Adjective Ratings for Roughness of Rigid Pavements:

Roughness Index Range	Adjective Rating
Less than 75	Very Smooth
76 - 90	Smooth
91 - 125	Slightly Rough
126 - 170	Rough
171 - 220	Very Rough
221 - or more	Unsatisfactory

6. An attempt was made to measure the strain in the concrete of CRC pavement. The concrete strainometers were showing about 100 micro-in./in. about 300 psi (2068 kPa) of tension when the nearby induced crack appeared. The tensile strain increased to about 160 micro-in./in. (about 480 psi) (3310 kPa) during the first winter.
7. The attempt to measure long-term strains in the CRC pavement was not very successful. Most of the strain gages failed during the first year. The limited long-term data available indicated that the strains in the steel tended to shift more toward compression with time. However, it is uncertain how much of the shift was due to drift in the instrumentation.
8. In general, the performance of CRC pavements (11 to 14 years of service) has been satisfactory. Few localized failures have developed and were found to be caused by the construction deficiencies, poor substructure drainage, and higher level of load applications. The amount of patching decreases as the slab thickness increases but increases as the depth of reinforcement and cumulated loading increases.

IMPLEMENTATION OF FINDINGS

The knowledge gained in preparing the plan details and construction specifications for the experimental pavements, together with the experience gained from the pavements' behavior, have provided the basis for development of the present CRC design standards and specifications being used by the Illinois Department of Transportation.

Seven-inch (178-mm) CRC pavement on granular subbase in a trench construction does not appear to have the structural integrity required to overcome the variety of construction imperfections or other deficiencies that sometimes contribute as a group to a premature failure of pavements carrying heavy truck loads. Based on these observations, the pavement design policies in Illinois have been revised to provide an 8-in. (203-mm) minimum thickness of CRC pavement with reinforcement placed 3 ± 1 in. (76 ± 25 mm) below the surface, over a stabilized subbase and having a positive underdrain system. For CRC pavements having a thickness greater than 8 inches (203 mm), the reinforcement bars are to be placed $3\ 1/2 \pm 1$ in. (89 ± 25 mm) below the pavement surface.

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